S-225-2007
(Consultant)

MEMORANDUM

TO: Allan Frank, P.E.
Division of Structural Design

FROM: William Broyles, P.E.
Geotechnical Branch Manager

BY: Bart Asher, P.E., P.L.S. BA
Geotechnical Branch

DATE: May 9, 2007

SUBJECT: **Jefferson County**
1200 056
Mars # 6554101D
Retaining Wall
S9280 (W65-10)
Ohio River Bridges Project
Kennedy Interchange – Section 1
Item No. 5-118.18 & .19

The geotechnical engineering report for this structure has been completed by Fuller, Mossbarger, Scott & May Engineers. We have reviewed and concur with the recommendations as presented in this report.

A copy of the report is attached. If you have any questions, please contact this office at 502-564-2374.

Attachment:

cc:	J. Callihan		
	R. Harris		
	N. Stroop		
	A. Calvin		
	B. Greene		
	B. Bryant		
	Glen Kelly	(KTA-QK4)	(w/o attachment)
	Adam Crace	(FMSM)	(w/o attachment)
	Donald Blanton	(FMSM)	(w/o attachment)

Report of Geotechnical Exploration
Retaining Wall S9280 (W65-10)

Ohio River Bridges Project
Kennedy Interchange - Section 1
Item Nos. 5-118.18 & 19
Jefferson County, Kentucky

Prepared for:
KTA-Qk4
Louisville, Kentucky

April 24, 2007

Fuller, Mossbarger, Scott & May Engineers, Inc.

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ENGINEERS

April 24, 2007

O.1.1.LX2004130-9280R01

Mr. Glen Kelly, PE
KTA-Qk4
815 Market Street, Suite 300
Louisville, Kentucky 40202

Re: Report of Geotechnical Exploration
Retaining Wall S9280 (W65-10)
Ohio River Bridges Project
Kennedy Interchange - Section 1
Item Nos. 5-118.18 & 19
Jefferson County, Kentucky

Dear Mr. Kelly:

Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) is submitting the geotechnical engineering report for the referenced retaining structure with this letter. The exploration described herein generally followed the guidelines presented in the Kentucky Transportation Cabinet's Geotechnical Manual and the Final Boring Plan dated February 28, 2006. This report also addresses comments offered by the Branch subsequent to their review of a draft copy of this report.

This report presents results of the field exploration along with our recommendations for the design and construction of the subject retaining wall. As always, we have enjoyed working with your staff and if we can be of further assistance, please contact our office.

Sincerely,

FULLER, MOSSBARGER, SCOTT AND MAY
ENGINEERS, INC.

Adam A. Crace, PE
Senior Project Engineer

Donald L. Blanton, PE
Project Manager

/rdr

cc: Bill Broyles, PE (KYTC - Geotechnical Branch) 3 Bound Copies and Electronic Files
ProjectWise

Report of Geotechnical Exploration
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Appendix

Report of Geotechnical Exploration
Retaining Wall S9280 (W65-10)

Ohio River Bridges Project
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Jefferson County, Kentucky

1. Introduction

1.1. Project Overview

The Bi-State Management Team, consisting of representatives from the Federal Highway Administration (FHWA), Kentucky Transportation Cabinet (KYTC) and Indiana Department of Transportation (INDOT), is planning and overseeing the design of the Ohio River Bridges Project, that will address the cross-river transportation needs in Louisville, Kentucky and Southern Indiana. The Ohio River Bridges Project consists of six (6) separate design sections.

- Section 1 - Kennedy Interchange
- Section 2 - Downtown Bridge
- Section 3 - Downtown Indiana Approach
- Section 4 - East End Kentucky Approach
- Section 5 - East End Bridge
- Section 6 - East End Indiana Approach

As a part of the Ohio River Bridges Project, the Kennedy Interchange will be reconstructed/relocated just south of its current location. The relocation includes the widening, reconstruction and construction of over 80 bridges, construction of approximately 28 retaining walls and about 22 miles of roadway, ramps and connectors to allow for more efficient traffic movement. Kentucky Transportation Associates (KTA), a collaboration of several engineering consulting firms, is serving as the design consultant for the Kennedy Interchange reconstruction/relocation.

1.2. Structure Location and Description

Reconstruction of the Kennedy Interchange section of the Ohio River Bridges Project includes widening of the existing interstate alignments to accommodate additional lanes of traffic and numerous new entrance, exit, and connector ramps planned to improve traffic flow. Design of the widened alignments and new ramps incorporates the use of retaining walls to address right-of-way constraints and limit encroachment upon adjacent property. This report specifically addresses the geotechnical concerns relative to the retaining wall designated as S9280 (W65-10). Project plans provided to Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) by KTA-American Consulting Engineers, PLC (American), indicate this retaining wall is proposed to accommodate the construction of Ramp 2. The planned wall is located on the southeast side of I-65 between bridge S0760 (B65-6), the Ramp 2 crossing over S0620 (BA-1), Jefferson Street, and Jackson Street and S0450 (B65-9), the I-65 crossing over East Market Street. The proposed wall alignment roughly parallels the existing interstate. The map provided in Appendix A illustrates the retaining wall site in relation to the planned project alignments and associated structures as well as the existing city streets and current interstate alignment. Appendix B presents structure drawings downloaded from the KTA ProjectWise website on March 1, 2007. The recommendations provided in this report are based on the wall configuration presented in these drawings.

Project plans indicate the wall is to be constructed between approximate Ramp 2 Stations 39+00 and 42+08, resulting in a length of approximately 308 feet. Cross-sections show the beginning of the wall to be situated near the toe of the existing interstate embankment and to climb the slope moving ahead-station and then moving back down the embankment near the bridge S0450 (B65-9). In addition, a shorter toe wall is positioned approximately 20 to 50 feet downslope from the proposed retaining wall. It is our understanding that this arrangement was selected to provide green space for plantings between the walls in order to satisfy aesthetic design requirements for the project. Structure plans indicate two options are being explored for the wall; (1) Mechanically Stabilized Earth (MSE) wall, and (2) Cast-in-Place (CIP) Cantilever Retaining Wall. The maximum wall height is on the order of 36.0 feet for the MSE option and 37.0 feet for the CIP option, as measured from the bottom of footing up to roadway grade.

Based on discussions with the Design Team, both the MSE and CIP alternates are being advanced for the subject retaining wall. Therefore, recommendations will be provided herein for each alternate, including separate geotechnical drawing sets in Appendix C and D for the MSE and CIP alternates, respectively. The recommendations provided in this report are based on wall geometries, heights and bearing elevations discussed herein. If roadway design modifications result in retaining wall geometries different than those discussed and evaluated herein, the Design Team should notify FMSM and provide the design changes for re-evaluation of the retaining wall systems and modification of the recommendations, as applicable.

2. Topography and Geologic Conditions

The project is located in the northwestern portion of Central Kentucky within the Outer Bluegrass Physiographic Region. The topography within the Outer Bluegrass varies from rolling hills to relatively flat, low-lying areas adjacent to major drainage features. The retaining wall site is located in downtown Louisville, approximately ¾-mile south of the Ohio River. As such, the Ohio River will influence groundwater levels at the proposed structure site. Topography within the vicinity of the bridge is relatively flat, with local relief generally less than five feet. However, highway embankments dissect the area and can rise as much as 35 feet above the surrounding terrain.

Available geologic mapping (<u>Geologic Map of Parts of the Jeffersonville, New Albany, and Charlestown Quadrangles</u>, Kentucky-Indiana, USGS, 1974) shows the structure site to be underlain by Outwash deposits of the Pleistocene geologic period. The mapping describes the Outwash as varying in thickness up to about 130 feet and consisting of sand, gravel, silt and clay deposited as alluvium by low-gradient rivers formed by glacial melt waters.

The geologic mapping does not depict structure contours within the immediate vicinity of the proposed retaining wall site because of insufficient data. However, structure contours drawn on the top of the Waldron Shale in the Jeffersonville Quadrangle and the base of the New

Albany Shale in the New Albany Quadrangle indicate the bedrock is relatively flat. The mapping shows the Springdale Anticline to be located approximately 3.8 miles southeast of the project, but does not note any faults or other detrimental geologic features to be present within the immediate vicinity of the structure site.

3. Drilling and Sampling Operations

FMSM developed a boring plan for the proposed retaining wall after a review of available structure plans, profiles, and roadway cross-sections provided by KTA. The original boring plan called for the advancement of five sample borings, designated herein as Hole Nos. 1W-27, 1W-77, 1W-78, 1W-368 and 1W-79. Section 4 of this report provides detailed discussion of the subsurface conditions encountered during the drilling program.

KTA – Qk4 survey personnel established the boring locations and surface elevations in the field in accordance with the Final Boring Plan dated February 28, 2006. Table 1 provides a summary of the stations, offsets, elevations, and depths of the borings drilled for the subject retaining wall (all measurements are expressed in feet). The boring locations presented in the table are referenced to I-65 stationing.

Table 1. Summary of Borings

Hole No.	Station/ Offset*	Surface Elevation	Top of Rock Elevation	Refusal/ Begin Core Elevation	Length of Core	Boring Termination Depth	Bottom of Hole Elevation
1W-27	657+58, 165?Rt.	462.9	--	NR (402.9)	--	60.0	402.9
1W-77	658+67, 157?Rt.	461.2	--	NR (401.2)	--	60.0	401.2
1W-78	659+84, 182?Rt.	462.6	--	NR (437.6)	--	25.0	437.6
1W-368	659+85, 114?Rt.	492.5	--	NR (417.5)	--	75.0	417.5
1W-79	661+01, 130?Rt.	462.2	--	NR (382.2)	--	80.0	382.2

* Station and Offset based on I-65 Centerline
NR indicates no refusal

FMSM personnel performed drilling and sampling operations in April and May of 2006. A geotechnical engineer from FMSM monitored the field operations and adjusted the boring program as field and/or subsurface conditions warranted. The drill crews advanced the borings for the subject wall utilizing a truck mounted drill rig equipped with hollow-stem augers. The field personnel generally performed soil sampling at five-foot intervals of depth to provide in situ strength data and specimens for subsequent laboratory strength and/or classification testing. Typically, undisturbed thin-wall (Shelby) tube samples were obtained within cohesive soil horizons and standard penetration (SP) testing was performed within granular (non-cohesive) materials. The drill crews checked each boring for the presence of groundwater prior to backfilling. The Subsurface Data Sheets in Appendixes C and D provide boring layouts that depict the locations of the borings in relation to the planned wall alignment as well as graphical logs presenting the results of the drilling, sampling, and laboratory testing programs. Refer to Appendix E for the Coordinate Data Submission Form summarizing the as-drilled boring locations, surface elevations, and associated latitudes and longitudes.

The drill rig utilized for the sampling operations was equipped with an automatic hammer to perform SP testing in accordance with Section 302-5 of the current KYTC Geotechnical Manual. The use of an automatic hammer provides for a more efficient and consistent transfer of energy than traditional SP testing with a safety hammer/rope/cat-head system. Thus, blowcounts observed from an automatic hammer are lower than those observed with the safety hammer system. Typical correlations for SP results used in geotechnical engineering practice are based on the safety hammer system and require that blowcounts from SP testing using an automatic hammer be corrected for efficiency. A discussion on the correction of the blowcounts is included in Section 6 of this report. The corrected N-values were used to derive strength and settlement parameters utilized in applicable engineering analyses.

4. Soil, Bedrock, and Groundwater Conditions

The drilling and sampling operations performed for the retaining wall indicate the subsurface materials consist of relatively thick (120+ feet) soil deposits consistent with the outwash/alluvial type materials described by the geologic mapping. In general, the subsurface materials observed during drilling operations primarily consist of a relatively thin mantle of clay overlying sand deposits extending to bedrock. Drilling operations from surrounding projects suggest the top of bedrock is about 125 feet below the ground surface.

Surface materials overlying the outwash deposits consist of topsoil, asphalt, concrete, crushed stone, and fill materials associated with interstate construction and previous development in the city of Louisville. Topsoil, approximately 0.4 feet in thickness, was observed at the top of Hole No. 1W-78, drilled along the wall alignment near the toe of the existing interstate embankment. Generally, the zone described as topsoil consisted of an organic dark brown soil mantle containing grass roots. Hole Nos. 1W-27 and 1W-77 did not encounter topsoil because the area had been stripped of topsoil for the current building construction. Drilling operations encountered asphalt underlain by a layer of crushed stone within Hole Nos. 1W-368 because the boring was located along the shoulder of the existing I-65 northbound. Boring No. 1W-79 encountered concrete underlain by a layer of crushed stone because the boring was located within the existing city sidewalk. Fill materials consisting of silty to sandy lean clay mixed with cobbles, brick fragments, and glass were observed in each boring drilled for the subject retaining wall. These materials were encountered beneath the cover material in Hole Nos. 1W-27, 1W-77, 1W-78, and 1W-79, and beneath the interstate embankment material within the hole drilled along the shoulder of I-65. The thickness of these fill materials was observed to be on the order of two to four feet.

Hole No. 1W-368 was positioned along the shoulder of the existing exit ramp and advanced through the roadway embankment to provide information concerning the existing fill material. The field engineer described the embankment material as shale shot rock. Loose sands and gravels, consistent with the outwash deposits encountered across the project area, were observed below this fill material.

The outwash deposits encountered within the test borings generally consisted of approximately 13 to 16 feet of sandy lean clay overlying relatively thick sand deposits (100+ feet) with varying amounts of gravel and silt. The field engineer visually described the clay soils as being brown to dark brown in color, damp to moist in terms of natural moisture content, medium stiff to stiff in consistency, and containing varying amounts of sand and gravel. The natural moisture content of the clay materials generally increased with increasing depth.

The sands observed in the borings are brown to gray in color, fine- to medium-grained, damp to wet in terms of natural moisture content, loose to dense, and contain varying amounts of gravel sized particles. Uncorrected N-values from SP testing ranged from a low of 3 to a high of 64 blows per foot. The upper 10 feet of the sand deposits encountered within Hole Nos. 1W-77, 1W-78 and 1W-79 can be described as loose and exhibit low N-values (10 or less), with an average uncorrected N-value of approximately 9. In general, the sand and gravel deposits are medium dense to dense with N-values ranging from a low of 11 to a high of 64 blows per foot (average uncorrected N-value of approximately 26).

FMSM personnel recorded an approximate measurement of the depth to the groundwater surface at each boring during drilling and sampling operations. Based on the groundwater level observations prior to backfilling the borings, the groundwater level at the structure site varies from approximate elevation 420.9 feet at the location of Hole No. 1W-79 to 435.8 feet at Hole No. 1W-77. The water level recorded at Hole No. 1W-77 seems to have been taken from a perched water table so that information should be discounted during the evaluation of this structure. The average elevation derived from the water levels taken at the time of drilling is 421.2 feet, which correlates well with the normal pool elevation of 420 feet for the Ohio River noted on the geologic mapping. The graphical logs provided on the Subsurface Data Sheets in Appendixes C and D depict the approximate location of the groundwater surface recorded in each boring, as applicable.

5. Laboratory Testing and Results

5.1. General

Selected soil specimens recovered during standard penetration testing and Shelby tube sampling operations were subjected to natural moisture content, wash gradation (silt plus clay determinations), soil classification, unconfined compressive strength, and one-dimensional consolidation testing. Laboratory personnel developed the soil classification identifications in accordance with both the Unified (USCS) and AASHTO soil classification systems.

Laboratory testing was performed in accordance with applicable American Association of State Highway Transportation Officials (AASHTO) or Kentucky Methods of soil testing specifications. The test results were used to establish material properties for subsequent engineering analyses to estimate soil bearing capacity and settlement of the proposed retaining wall options, as well as evaluate the retaining wall stability. The following paragraphs provide detailed discussions of the laboratory testing program

5.2. Testing of Cohesive Soils/Undisturbed (Shelby) Tube Testing

The borings drilled for the subject wall included undisturbed (Shelby) tube sampling within predominantly cohesive soil horizons. FMSM?s soils laboratory extruded the tubes and trimmed six-inch specimens. Lab personnel determined visual descriptions, unit weights (wet and dry), and natural moisture for each six-inch specimen prior to submitting a summary of the extruded specimens to a geotechnical engineer for assignment of lab testing. The laboratory testing performed on the extruded samples consisted of engineering classification, unconfined compressive strength, and one-dimensional consolidation testing. The following paragraphs provide further discussion of the test results.

5.2.1. Engineering Classification Test Results for Cohesive Samples

FMSM performed engineering classification testing on selected six-inch Shelby tube specimens. The testing generally included one classification test per soil type in a Shelby tube. The cohesive soils primarily classify as SC with one occurrence of CL according to USCS, and as A-6 with lesser occurrences A-2-4 based on the AASHTO classification system. Testing of the Shelby tube samples encountering the top of the sand deposits resulted in classifications of SW-SM based on the USCS and A-1-b based on the AASHTO classification system. The Subsurface Data Sheets provided in Appendixes C and D depict the results of the classification testing adjacent to the graphical logs.

5.2.2. Unconfined Compressive Strength Testing of Cohesive Samples

Unconfined compressive strength testing was performed to provide information from which soil strength parameters could be estimated. The unconfined compressive strength values range from 1,580 psf (0.79 tsf) to 2,180 psf (1.09 tsf). The results of the unconfined compressive strength tests are presented next to the sample borings on the geotechnical drawings in Appendixes C and D and are summarized in Table 2.

Table 2. Summary of Unconfined Compressive Strength Tests

Hole No.	Station and Offset	Sample Interval (ft)	Dry (pcf)	Wet (pcf)	Moisture Content %	Unconfined Compressive Strength (psf)	Estimated Cohesion (psf)
1W-78	659+84, 182’ Rt.	5.1 – 5.6	104.4	123.8	18.6	1,580	790
1W-78	659+84, 182’ Rt.	10.9 – 11.4	103.9	126.1	21.4	2,180	1,090

The unconfined compressive strength can be used to estimate the bearing capacity and cohesion of a soil material. The value of cohesion in an engineering analysis is generally estimated to be one-half of the unconfined compressive strength for cohesive soils. Based on the above test results, the cohesion values derived from unconfined compression strength testing range from 790 psf (0.40 tsf) to 1,090 psf (0.55 tsf).

5.2.3. One-Dimensional Consolidation Testing

FMSM’s laboratory performed one-dimensional consolidation testing on a selected sample extruded from the Shelby tubes to provide initial void ratio and consolidation parameters utilized in settlement analyses. The results of the consolidation tests are summarized in Table 3 and are presented in Appendix F.

Table 3. Summary of One-Dimensional Consolidation Tests

Hole No.	Station and Offset	Test Interval (ft)	Initial Void Ratio (e_o)	Compression Index (C_c)	Recompression Index (C_r)	Pre-Consolidation Pressure (P_c) (psf)
1W-78	659+84, 182? Rt.	5.0 – 7.0	0.765	0.198	0.033	1,000

5.3. Laboratory Testing of Non-Cohesive Soils/Standard Penetration Test Samples

In general, recovered soil specimens from SP testing were subjected to natural moisture content and silt plus clay determinations. However, in lieu of silt plus clay determinations, selected samples were combined for engineering classification testing. The SP samples tested classify primarily as SM and CL with lesser occurrences of SP, SW, SW-SM, SP, SM, GP-GM, GW-GM, SC and GC-GM according to USCS, and primarily as A-1-b with lesser occurrences of A-1-a, A-4, A-6 and A-3 based on the AASHTO classification system. Refer to Table 4 for a summary of the classification testing performed on soil samples recovered from SP testing.

Table 4. Summary of Non-Cohesive Soil Classification Testing

USCS		AASHTO	
Soil Type	Percentage	Soil Type	Percentage
SM	15	A-1-b	42
CL	15	A-1-a	26
SP	11	A-4	16
SW	11	A-6	11
SW-SM	11	A-3	5
SP-SM	11		
GP-GM	11		
GW-GM	5		
SC	5		
GC-GM	5		

The results of the classification testing were used in conjunction with the N-values from SP testing to estimate soil strength and settlement parameters based on published correlations of such data.

6. Derivation of Soil Parameters

6.1. Correction of Standard Penetration Test Data

As discussed in Section 3 of this report, drilling and sampling operations utilized a drill rig equipped with an automatic hammer to perform SP testing. Standard correlations for SP testing consider blowcounts using a safety hammer/rope/cat-head system, generally estimated to be 60 percent efficient. Thus, correlations are based upon what is currently termed as N_{60} data. The efficiency of the automatic hammer used for this exploration was estimated to be approximately 80 percent based on previous efficiency testing of FMSM drill rigs equipped with such equipment. The correction for hammer efficiency is a direct ratio of relative efficiencies as follows:

$$N_{60} = N_{80}\left(\frac{80}{60}\right) \tag{6.1}$$

FMSM corrected standardized N_{60} values for the effect of overburden pressure prior to using the data in conjunction with correlations for non-cohesive soil parameters. N_{60} values were normalized to vertical effective overburden stresses of 2,000 pounds per-square foot. This calculation requires an effective unit weight for each soil horizon multiplied by the depth of the soil horizon. Liao and Whitman, as referenced in Seed and Harder [1990], proposed a relationship between the correction factor, C_N, and the effective overburden stress, σ':

$$C_N = \frac{1}{\sqrt{\sigma'}} \tag{6.2}$$

where:

C_N = correction factor for overburden stress

σ' = vertical effective overburden stress (tsf)

Consequently, the standardized corrected N-value, $(N')_{60}$ is equal to:

$$(N')_{60} = C_N N_{60} \tag{6.3}$$

where:

C_N = correction factor for overburden stress

N_{60} = standardized N-value

Appendix G contains summaries of the SP data and corrections for the five borings performed along the wall alignment. The spreadsheets also include correlations of corrected SP data with published correlations for estimates of unit weight and shear strength parameters. The values of $(N')_{60}$ were utilized to obtain relative densities, D_r, based on relationships developed by Tokimatsu and Seed [1988]. NAVFAC [1982] presents a relationship using relative density of specific soil types to correlate angle of internal friction, unit weight, and void ratio. Soil classifications for the correlations came from actual laboratory test results and visual observations, and were used to estimate an in situ unit weight of the material. Once the relationships for the angle of internal friction, unit weight and void ratio were established, an in situ unit weight was calculated based upon the natural moisture content.

6.2. Development of Soil Profile

FMSM derived subsurface characterizations for the foundation soils along the wall alignment based upon the results of the drilling and sampling program discussed in Sections 3 and 4 of this report, and the laboratory testing addressed in Section 5. The division of soil horizons was based on visual soil descriptions, laboratory classification data, and corrected SP data associated with Boring Nos. 1W-27, 1W-77, 1W-78, 1W-368, and 1W-79. The Subsurface Data Sheets in Appendixes C and D present the subsurface profile and summaries of estimated soil parameters modeled in engineering analyses.

A geotechnical engineer derived estimated soil parameters for each soil horizon. Strength and settlement parameters for the cohesive materials were estimated based on the results of laboratory classification, unconfined compressive, and one-dimensional consolidation testing. Laboratory test results were used from nearby borings from adjacent structures when necessary. The parameters derived for the cohesive materials are representative of sandy lean clay soils and are typical of clay soils found in this region of the state. Likewise, the settlement and strength parameters for the non-cohesive materials (sand deposits) were estimated based on corrected SP data, laboratory classification testing, and correlations of such data. Values of internal angles of friction (ϕ?) for granular soils obtained from the correlations vary from 30.0 to 41.0 degrees. A review of these parameters indicate in general an increasing trend with depth which coincides with dense coarse grained deposits typically found within the site's geological setting.

At the writing of this report, a borrow source for embankment material has not been identified. Thus, it has been estimated that the new embankment material will exhibit strength properties similar to the material comprising the clay portion of the existing roadway embankment. Laboratory testing of the existing clay embankment materials and alluvial clay foundation soils yielded effective internal friction angles varying from 20 to 41 degrees and effective cohesion values ranging from 0 to 725 pounds per square foot. A few of the tests resulted in values higher than are normally associated with sandy lean clay soils. The results of this testing were tempered with experience and engineering judgment when selecting representative values for evaluation of the wall options. The shear-strength parameters modeled for retaining wall and slope stability analyses are more typical of clay soils in the project area and are summarized in Table 5 below.

Table 5. Modeled Embankment Shear Strength Parameters

Embankment Material		Retained Fill	
Total Stress	Effective Stress	Total Stress	Effective Stress
c = 1400 psf ϕ = 0° γ = 120 pcf	$\bar{c}$ = 200 psf $\bar{\phi}$ = 23° γ = 120 pcf	c = 1400 psf ϕ = 0° γ = 120 pcf	$\bar{c}$ = 170 psf $\bar{\phi}$ = 27° γ = 120 pcf

The lower shear strengths modeled for embankment materials are representative of relatively weak clays that are known to exist in the project area. Non-durable shales in the Louisville area are known to weather to clay soils exhibiting effective friction angles on the order of 23 degrees. The higher shear strength parameters modeled for the retained fill were derived based on the results of laboratory testing conducted on existing clay embankment materials and alluvial foundation soils. It should be noted that confirmation testing of borrow source materials will be required to verify that the materials exhibit minimum strengths equal to or greater than those outlined above. Recommendations for confirmation testing of borrow material are provided in Section 11 of this report.

7. LRFD Retaining Wall Load and Resistance Factors

7.1. Selection of LRFD Load and Resistance Factors

The KYTC has mandated that the Kentucky portion of the Ohio River Bridges project will use Load and Resistance Factor Design (LRFD) Methodology for design of project structures. LRFD is a design approach in which applicable failure and serviceability conditions can be evaluated considering the uncertainties associated with loads and materials resistances. In general, the engineering analyses performed for evaluation of the retaining wall options followed the current AASHTO LRFD guidelines.

LRFD methodology incorporates the use of load factors and resistance factors to account for uncertainty in applied loads and load resistance of structure elements separately in contrast to the Factor of Safety traditionally applied only to the resistances in Allowable Stress Design (ASD) methodology. The AASHTO LRFD Bridge Design Specifications outline load factors and combinations for various strength, extreme event, service, and fatigue limit states. Section 11 of the AASHTO Specifications, the section of the code governing retaining walls, mandates the evaluation of bearing resistance failure, lateral sliding, and excessive loss of base contact (overturning) at the strength limit state and excessive vertical displacement, excessive lateral displacement, and overall stability at the service limit state. Table 6 outlines the load factors used in evaluation of the retaining wall options.

Table 6. LRFD Load Factors for Retaining Wall Analyses

Load		Load Factors (γ)*		
		For Bearing Resistance Strength IA	For Sliding and Eccentricity Strength IB	For Settlement Service I
Dead Load of Structural Components	DC	1.25	0.90	1.00
Vertical Earth Pressure Load**	EV	1.35	1.00	1.00
Horizontal Earth Pressure Load	EH	1.50	1.50	1.00
Live Load Surcharge	LS	1.75	1.75	1.00

* From AASHTO LRFD Bridge Design Specifications, Fourth Edition, Tables 3.4.1-1 and 3.4.1-2

** From Dead Load of Earth Fill

Selection of LRFD resistance factors account for the type of loading (sliding versus bearing) and the variability and reliability of models or methodologies used to determine nominal resistance (R_n) capacities. The AASHTO LRFD Bridge Design Specifications outline recommended resistance factors for standard static analysis methodologies, industry accepted methodologies for field verification, and levels of construction quality control. The selection of resistance factors used in evaluation of the retaining wall options is further discussed in the following sections of the report, as applicable.

7.2. LRFD Evaluation Criteria

The AASHTO LRFD Bridge Design Specifications outline in Section 11 geotechnical criteria to analyze the external stability of a retaining wall. As with the traditional ASD method of designing retaining walls LRFD also considers the bearing capacity, overturning, sliding, and global stability in the design process. The bearing capacity is evaluated by checking to see that the calculated bearing capacity of the local soils is greater than the induced load of the wall from the Meyerhof Uniform Bearing Pressure. The overturning is checked by evaluating the eccentricity by checking to see that the resultant of the reaction forces shall be within the middle one half of the base width, which corresponds to an eccentricity of 25 percent of the base width (0.25B). Sliding is evaluated by Capacity-Demand Ratio for Sliding ($CDR_{Sliding}$) which is equal to the factored capacity divided by the factored load. The $CDR_{Sliding}$ should be greater than or equal to 1.0. Presently LRFD methodology does not translate well to traditional slope stability analyses. This in conjunction with the KYTC typically recommending more stringent minimum requirements for slope stability. Therefore, FMSM evaluated global stability in terms of traditional ASD methodology using factors of safety. The KYTC Geotechnical Manual recommends minimum target factors of safety of 1.2 and 1.6 for short- and long-term global slope stability analyses, respectively, performed at structure locations.

8. Evaluation of Mechanically Stabilized Earth Wall Option

8.1. General

The MSE wall configuration evaluated for the subject retaining structure was developed based on plan view and profile drawings downloaded from the KTA ProjectWise website on March 1, 2007. Based on discussions with the Design Team, FMSM revised the bearing elevation of the wall to provide a minimum embedment of two feet and to incorporate steps, as practical for construction, to reduce the wall area. Appendix C presents a plan view and Subsurface Data Sheets for the MSE option. Table 7 summarizes the station limits and wall configurations evaluated for the MSE wall. The wall heights outlined below are as measured from the planned bearing elevation up to the top of the retained fill. The backfill slope behind the wall will be level.

Table 7. Summary of Wall Configuration Evaluated for MSE Option

Alignment	Station Limits	Maximum Wall Height	Base of Wall Elevation
Ramp 2	39+00 to 40+00	36.0 ft	460.0 ft
Ramp 2	40+00 to 40+50	24.0 ft	470.5 ft
Ramp 2	40+50 to 41+00	17.5 ft	476.5 ft
Ramp 2	41+00 to 41+50	12.0 ft	481.5 ft
Ramp 2	41+50 to 42+08	26.5 ft	467.5 ft

FMSM performed engineering analyses to estimate bearing capacity and potential settlements along the wall profile, and evaluate retaining wall and slope stability. The analyses are based on wall geometries, heights and bearing elevations discussed herein. If roadway design modifications result in retaining wall geometries different than those discussed and evaluated herein, the Design Team should notify FMSM and provide the design changes for re-evaluation of the retaining wall, as applicable. The analyses performed to evaluate the MSE option are discussed further in the following sections.

8.2. External Stability

FMSM evaluated the external stability of the MSE wall option based on the wall heights and planned bearing elevations outlined in Table 7. The MSEW computer program (Version 3.0) was used to evaluate sliding stability and eccentricity of the planned MSE configuration as well as determine the Meyerhof uniform bearing pressure applied to the foundation materials. The MSEW computer program, developed by ADAMA Engineering, Inc., uses AASHTO LRFD design methodology in conjunction with the AASHTO 2002 and NHI-043 design guidelines for MSE wall design.

As discussed in Section 7 of this report, the selection of resistance factors accounts for the type of loading (sliding versus bearing capacity) and the variability and reliability of models or methodologies used to determine nominal resistance (R_n) capacities. Table 8 summarizes the resistance factors for sliding stability outlined in the AASHTO LRFD Bridge Design Specifications based on the wall materials and bearing medium.

Table 8. LRFD Resistance Factors for Sliding Stability

Bearing Condition	Resistance Factor* (φ_τ)
Precast Concrete placed on Sand	0.90
Cast-In-Place Concrete on Sand	0.80
Cast-In-Place or Precast Concrete on Clay	0.85
Soil on Soil	0.90

* From AASHTO LRFD Bridge Design Specifications, Fourth Edition, portion of Table 10.5.5.2.2-1

Because the MSE option consists of a reinforced soil volume bearing on clay foundation soils or granular replacement material, the resistance factor modeled in sliding stability analyses is 0.90, corresponding to a soil on soil bearing condition.

The soil parameters and subsurface profile modeled in the wall analyses were derived based on the drilling, sampling, and lab testing programs discussed in previous sections of this report. Refer to Section 6 for a discussion of the derivation of soil parameters and development of the subsurface profile. Table 9 summarizes the soil parameters modeled in the wall analyses.

Table 9. Soil Parameters Modeled in MSE Wall Analyses

Material	Parameter
Retained Fill	Φ = 27° Unit Weight = 120 pcf
Reinforced Fill (Reinforced Soil Volume)	Φ = 34° Unit Weight = 115 pcf
Clay Foundation Soils (Material Beneath the Wall Footprint)	Φ = 32° Unit Weight = 128 pcf
Granular Embankment – Crushed Stone (Material used for Replacement of Over Excavated Foundation Soils)	Φ = 38° Unit Weight = 120 pcf

Φ = internal friction angle

FMSM performed external stability analyses at select locations along the wall alignment incorporating the load factors outlined in Table 6, as applicable. Analysis of the walls included the application of a live load surcharge in accordance with KYTC and AASHTO recommendations for walls subjected to traffic loading. The magnitude of the surcharge varies from two to five feet, based on the wall height and distance between the back of the wall and lanes of travel (Table 3.11.6.4-2 of the AASHTO LRFD Bridge Design Specifications). For wall heights equal to or greater than 20 feet, the applicable surcharge load is 2 feet of soil. Likewise, the applicable surcharge load for a wall ten feet in height is 3.5 feet of soil. Thus, interpolating between the recommended surcharge load values (based on wall height) and using a unit weight of soil equal to 120 pcf, the surcharge loads modeled in the analyses performed for the subject retaining structure are 240 psf for wall heights of 24.0 feet, 26.5 feet and 36.0 feet. The surcharge load was modeled at 285 psf and 384 psf for wall heights of 17.5 feet and 12.0 feet, respectively. Table 10 summarizes the results of the MSE wall analyses performed for the subject retaining structure.

Table 10. Summary of MSE Wall Analyses

Station Interval*	Max Wall Height	Strap Length	$CDR_{Sliding}$	Eccentricity	Meyerhof Uniform Pressure
39+00 to 40+00	36.0 ft	25.2 ft = 0.7H	1.28	6.02 ft = 0.24B	6,130 psf
40+00 to 40+50	24.0 ft	19.2 ft = 0.8H	1.38	3.80 ft = 0.20B	5,630 psf
40+50 to 41+00	17.5 ft	14.0 ft = 0.8H	1.31	3.16 ft = 0.23B	4,480 psf
41+00 to 41+50	12.0 ft	10.8 ft = 0.9H	1.34	2.53 ft = 0.23B	3,400 psf
41+50 to 42+08	26.5 ft	21.2 ft = 0.8H	1.41	4.11 ft = 0.19B	8,955 psf

* Ramp 2 Stationing

FMSM initially estimated the reinforcement strap length modeled in the MSE wall analyses to be 70 percent of the wall height (0.7H), with a minimum strap length of eight feet. However, the strap lengths were increased to 80 percent of the wall height (0.8H) for walls less than 29 feet in height and to 0.9H for walls less than 15.0 feet in height because the strap lengths did not meet the LRFD eccentricity requirements.

8.3. Bearing Capacity Analyses of the Existing Soils

Based upon the information derived from drilling, sampling, and laboratory testing operations conducted along the planned wall alignment, nominal bearing capacity estimates were performed for comparison with the induced wall loadings. The methodology used to calculate the nominal bearing capacity (q_n) is presented in the AASHTO LRFD Bridge Design Specifications, Fourth Edition, Section 10.6.3 and the US Army Corps of Engineers “Bearing Capacity of Soils”, EM 1110-1-1905.

Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on fine-grained (clayey) alluvial soils between Stations 39+00 and 40+00 and shale (shot rock) embankment materials between Stations 40+00 and 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clayey materials from Stations 39+00 to 40+00. Cohesion values of 790 psf and 1,090 psf derived from unconfined compression test results and correlations of corrected SP N-values yields a nominal bearing capacity on the order of 4,225 psf for the clay alluvium. Since the remainder of the wall will be founded on the existing shale (shot rock) embankment the bearing capacity will be controlled by the friction angle of the shale embankment. A friction angle of 35 degrees was derived from correlations of the corrected SP N-values and yields nominal bearing capacities from 27,050 psf to 44,310 psf.

When applicable, the nominal bearing capacity was adjusted to incorporate an increase (or decrease, as applicable) in bearing capacity for over excavation and replacement and for two-layered soil systems. The nominal bearing capacity of the foundation soils was increased for punching resistance through the granular replacement material based on methods outlined in the US Army Corps of Engineers reference. FMSM only included the contribution of punching resistance to the bearing capacity when requiring the use of biaxial geogrid as part of the granular replacement material. The calculation of bearing capacity for a two-layer soil system is based on the methodology outlined in the AASHTO LRFD Bridge Design Specifications and U.S. Army Corps of Engineers reference.

It is FMSM?s understanding that the resistance factors outlined for bearing capacity in the AASHTO LRFD Bridge Design Specifications were calibrated for rigid footings and do not necessarily apply for MSE walls. AASHTO?s 17th Edition and "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes", FHWA publication number NHI-00-043, provide guidance on using a factor of safety of 2.0 for bearing capacity of MSE walls. Based on discussions with the FHWA and KYTC, a resistance factor for the bearing capacity of MSE walls was determined by using the predominant load factor divided by the factor of safety (1.35/2.0 = ± 0.67). Using a resistance factor of 0.67, the factored bearing capacity for the clay alluvium is on the order of 2,830 psf.

Review of the planned bearing elevation and subsurface profile developed based on the drilling program indicate the wall will bear on existing shale (shot rock) embankment materials between Stations 40+00 and 42+08. However, the bearing elevation for the base of the wall is positioned near the weaker clay foundation soils so a reduction in the bearing capacity of the embankment materials will be necessary. Using the method for a two-layer soil system outlined in Section 10.6.3.1.2d of the AASHTO Specifications and a resistance factor (φ_b) of 0.67, FMSM developed Table 11 which outlines the factored bearing capacity applicable for various steps in retaining wall.

Table 11. Factored Bearing Capacity for MSE Wall Option

Station Interval**	Factored Bearing Capacity (q_R)* (psf)
40+00 to 40+50	5,730
40+50 to 41+00	8,870
41+00 to 41+50	16,220
41+50 to 42+08	4,180

* Using a Resistance Factor (φ_b) of 0.67.
** Ramp 2 Stationing

A review of the Meyerhof uniform pressure/required bearing capacity values determined for the MSE wall option as presented in Table 10 indicates the applied bearing pressures are greater than the factored bearing capacity for the clay alluvium between Stations 39+00 and 40+00 and for the shale (shot rock) embankment between Stations 41+50 and 42+08. As such, construction of the MSE wall option bearing directly on the in situ soils within the intervals mentioned above without some type of foundation soil modification will likely experience bearing capacity failure. FMSM recommends excavation of the foundation materials and replacement with Granular Embankment (crushed stone) as a means of spreading out the load exerted by the wall over a larger area, and thereby reducing the soil contact pressures to acceptable values. The estimated excavated area requiring granular embankment will vary as shown in Table 12 below. In addition, the interval between Stations 41+50 and 42+08 will also require additional horizontal over excavation of five feet beyond the wall perimeter and the use of geogrid reinforcement to assist in transferring wall loads to the foundation soil. Table 12 provides a summary of the over excavation and replacement requirements based on the anticipated wall loading and factored bearing capacity of the clay foundation soils.

Table 12. Summary of Over Excavation and Replacement for MSE Wall Option

Station Interval*	Maximum Wall Height	Approximate Bearing Elevation	Over Excavation	
			Vertical	Horizontal
39+00 to 40+00	36.0 ft	460.0 ft	8 ft (452.0 ft)	8 ft**
40+00 to 40+50	24.0 ft	470.5 ft	NA	NA
40+50 to 41+00	17.5 ft	476.5 ft	NA	NA
41+00 to 41+50	12.0 ft	481.5 ft	NA	NA
41+50 to 42+08	26.5 ft	467.5 ft	5 ft (462.5 ft)	5 ft**

* Ramp 2 Stationing
** Requires the use of geogrid reinforced Granular Embankment

Section 11 of this report provides recommendations further outlining specific details and locations of foundation soil modifications.

8.4. Settlement Analyses

FMSM performed settlement analyses at select locations in order to develop an estimated settlement profile along the wall alignment. Based on the planned bearing elevations and over excavation depths previously discussed, it appears that the wall will bear on both gravelly embankment materials and alluvial clay foundation soils. Settlement parameters for the embankment materials and foundations soils were estimated based on the results of the previously discussed drilling, sampling, and laboratory testing programs. Consolidation parameters for the clay type soils were derived from the results of one-dimensional consolidation testing. Settlement parameters for the granular (non-cohesive) materials were estimated based on corrected N-values correlated with laboratory classification testing as outlined in the guidelines presented in the FHWA Soil and Foundations Workshop Manual – Second Edition, pages 168 through 170. The estimated settlement parameters derived for each soil horizon are shown on the Subsurface Data Sheets presented in Appendix C.

The applied pressures used in the analyses were based on the LRFD Service I load combinations and the resulting Meyerhof uniform pressure distribution beneath the wall using previously discussed traffic surcharge load. The results of the analyses indicate the potential for up to approximately 8.7 inches of settlement of the soils beneath the MSE wall. Table 13 presents a summary of the settlement analyses performed for the subject MSE option. The settlement profile for the MSE wall is presented in Appendix C.

Table 13. Summary of Settlement Calculations for the MSE Option

Settlement Point Location*	Total Settlement (in.)	(ft)	Differential Settlement (ft)	Estimated Distance Over Which Settlement Occurs (ft)	Ratio of Differential Settlement
39+00	5.0	0.415			
			0.153	100	1 / 653
40+00	6.8	0.568			
			0.103	5	1 / 49
40+00	8.1	0.671			
			0.056	50	1 / 893
40+50	8.7	0.727			
			0.349	5	1 / 14
40+50	4.5	0.378			
			0.015	50	1 / 3,333
41+00	4.7	0.393			
			0.128	5	1 / 39
41+00	3.2	0.265			
			0.031	50	1 / 1,613
41+50	2.8	0.234			
			0.395	5	1 / 13
41+50	7.5	0.629			
			0.108	58	1 / 537
42+08	6.2	0.521			

* Ramp 2 Stationing

Settlement was generally estimated at “step” locations/changes in excavation depths. For the purpose of calculating a ratio of differential settlement, it was estimated that the differential settlement would occur over a distance of five feet at the step locations, otherwise the distance between settlement points was used.

At the time of this writing, it is FMSM?s understanding that the MSE walls will be constructed, allowed to settle, and then the permanent wall fascia will be attached. If the construction schedule does not allow this to occur, the wall Designer should consider the affects of differential settlement on the fascia. AASHTO LRFD Bridge Design Specifications and FHWA literature "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines" suggests that where significant differential settlements are anticipated (ratio of differential settlements greater than 1/100), sufficient joint width and/or slip joints must be provided to reduce the potential of panel cracking. Based on the settlement calculations presented in Table 13, all the step locations exhibit differential settlements greater than 1/100. The wall Designer should select the panels and size the joint widths and/or place slip joints between wall panels to accommodate the anticipated settlements. If this cannot be done, ground improvement techniques such as additional excavation of soil and replacement with select embankment, or the use of stone columns, geopiers, or other ground improvements techniques may be warranted to reduce the anticipated settlement.

FMSM performed time rate of settlement calculations for the planned wall configuration. Based on these calculations, it is estimated that 90 percent of the primary consolidation of the clay foundation soils will occur in about 189 days (27 weeks). Section 11 of this report includes recommendations for the installation and monitoring of settlement platforms.

8.5. Lateral Squeeze

Studies conducted by the FHWA have shown that walls bearing on deposits of compressible soils may experience horizontal deformations and/or movement. The condition causing the structural deformation is the unbalanced fill loading on each side of the wall, which causes the compressible foundation soils to move (squeeze) laterally.

FHWA publication NHI-00-43, "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes", suggests the potential for lateral squeeze, or horizontal deformation, exists if the pressure applied by a wall is greater than three times the undrained shear strength of the foundation soils. Based on the subsurface exploration program, the cohesive soil horizons extend along the entire length of the retaining wall. A design value of 790 psf was derived for this alluvial clay layer, from the test data obtained for this wall. The pressure increase at the middle of the alluvial clay layer resulting from wall loading is approximately 4,500 psf between Stations 39+00 and 40+00 using the Service I load combination. Based on the noted criteria, the pressure applied by the wall does exceed three times the undrained shear strength of the clay foundation soils (3C = 3 x 790 = 2,370 psf) indicating that the potential potential for lateral squeeze exists for the MSE wall option and should be considered in the design of the wall foundation system. The FHWA "Soils and Foundation Workshop Manual" suggests that the anticipated lateral movement may be estimated as 25 percent of the fill settlement. A settlement analysis was conducted at Station 39+00 and yielded an estimated settlement of 3.9 inches. Thus, the lateral deformation of the wall is estimated to be on the order of 1.0 inch. For the remainder of the retaining wall between Station 40+00 and 42+08 the bearing pressure increase based on the Service I load combination at the middle of the clay layer is less than three times the undrained strength of the clay foundation soils so the potential for lateral squeeze is low.

8.6. Global Slope Stability

FMSM evaluated the global stability of the anticipated roadway embankment/MSE wall configuration utilizing the REAME (Rotational Equilibrium Analysis of Multi-Layered Embankments) 2004 slope stability program, developed by Dr. Y.H. Huang at the University of Kentucky. The program estimates a circular (rotational) failure surface and calculates the factor of safety based on the Simplified Bishop method of slices. Short-term analyses using total-stress shear-strength parameters for the foundation and embankment materials simulate conditions that will exist immediately following the construction of the embankment. Long-term analyses, using effective-stress shear-strength parameters, simulate conditions that will exist long after the embankment is constructed and excess pore pressures within the materials have dissipated. Table 14 presents a summary of the slope stability analyses performed for the MSE wall option.

Table 14. Summary of Global Slope Stability Analyses for MSE Option

Location	Global Slope Stability	
	Short Term	Long Term
Ramp 2 Station 39+00, right of centerline	2.0	2.4

FMSM evaluated global stability in terms of traditional ASD methodology using factors of safety. The KYTC Geotechnical Manual recommends minimum target factors of safety of 1.2 and 1.6 for short- and long-term global slope stability analyses, respectively, performed at structure locations. Based on a comparison of the KYTC minimum target factors of safety and the results of the global stability analyses summarized in Table 14, the calculated factors of safety exceed the recommended minimums. Subsurface Data Sheet 6 of 6 in Appendix C presents results of the slope stability analyses, including predicted minimum factors of safety, predicted failure surfaces, and modeled groundwater table positions.

9. Evaluation of Cast-in-Place Retaining Wall Option

9.1. General

The CIP wall configuration evaluated for the subject retaining structure was also developed based on plan view and profile drawings downloaded from the KTA ProjectWise website on March 1, 2007. The plans indicate the CIP option exhibits a cantilever type configuration. As with the MSE option, FMSM revised the bearing elevation of the wall to provide a minimum of one foot of soil cover over the top of the wall footing and to incorporate steps, as practical for construction, to reduce the wall area. Appendix D presents a plan view and Subsurface Data Sheets for the CIP option. Table 15 summarizes the station limits and wall configurations evaluated for the CIP wall. The wall heights outlined below are as measured from the base of the wall footing up to the top of the retained fill. The backfill slope behind the wall will be level.

Table 15. Summary of Wall Configuration Evaluated for CIP Option

Alignment	Station Limits	Maximum Wall Height	Base of Wall Elevation
Ramp 2	39+00 to 40+00	37.0 ft	459.0 ft
Ramp 2	40+00 to 40+50	25.0 ft	469.5 ft
Ramp 2	40+50 to 41+00	18.5 ft	475.5 ft
Ramp 2	41+00 to 41+50	13.0 ft	480.5 ft
Ramp 2	41+50 to 42+08	26.5 ft	467.5 ft

FMSM performed engineering analyses to estimate bearing capacity and potential settlements along the wall profile, and evaluate retaining wall and slope stability. The analyses are based on wall geometries, heights and bearing elevations discussed herein. If roadway design modifications result in retaining wall geometries different than those discussed and evaluated herein, the Design Team should notify FMSM and provide the design changes for re-evaluation of the retaining wall, as applicable. The analyses performed to evaluate the CIP option are discussed further in the following sections.

9.2. External Stability

FMSM evaluated the external stability of the CIP wall option based on the wall heights and planned bearing elevations outlined in Table 15. The Florida Wall computer program (Version 2.05) was used to evaluate sliding stability and eccentricity of the planned CIP configuration as well as determine the pressures applied to the foundation materials. The Florida Wall program was developed by the Florida Department of Transportation and is based on a MathCAD® worksheet. FMSM modified the worksheet to use Coulomb earth pressure theory instead of Rankine, calculate sliding resistance based on the sliding friction angle between the base of the wall and the foundation materials, and to calculate eccentricity directly in addition to calculating the location of the resultant force. The worksheet was also modified to calculate maximum toe and minimum heel pressures based on current LRFD guidelines.

As discussed in Section 7 of this report, the selection of resistance factors accounts for the type of loading (sliding versus bearing capacity) and the variability and reliability of models or methodologies used to determine nominal resistance (R_n) capacities. Refer to Table 8 for a summary of the resistance factors for sliding stability outlined in the AASHTO LRFD Bridge Design Specifications based on the wall materials and bearing medium. Because the CIP option involves cast-in-place concrete bearing on the foundation materials, a resistance factor of 0.85 applies for evaluation of a wall bearing on the alluvial clay foundation soils and 0.80 applies for bearing on granular replacement material.

The soil parameters and subsurface profile modeled in the wall analyses were derived based on the drilling, sampling, and lab testing programs discussed in previous sections of this report. Refer to Section 6 for a discussion of the derivation of soil parameters and development of the subsurface profile. Table 16 summarizes the soil parameters modeled in the CIP wall analyses.

Table 16. Soil Parameters Modeled in CIP Wall Analyses

Material	Parameter
Retained Fill	Φ = 27° Unit Weight = 120 pcf δ = 17°
Clay Foundation Soils (Material Beneath the Wall Footprint)	Φ = 32° Unit Weight = 128 pcf δ = 17°
Granular Embankment – Crushed Stone (Material used for Replacement of Over Excavated Foundation Soils)	Φ = 38° Unit Weight = 120 pcf δ = 29°

Φ = internal friction angle
δ = interface friction angle between dissimilar materials (sliding friction angle)

FMSM performed external stability analyses at select locations along the wall alignment incorporating the load factors outlined in Table 6, as applicable. As with the evaluation of the MSE walls, analysis of the CIP option included the application of a live load surcharge in accordance with KYTC and AASHTO recommendations for walls subjected to traffic loading. For wall heights equal to or greater than 20 feet, the applicable surcharge load is 2 feet of soil. Likewise, the applicable surcharge load for a wall ten feet in height is 3.5 feet of soil. Thus, interpolating between the recommended surcharge load values (based on wall height) and using a unit weight of soil equal to 120 pcf, the surcharge loads modeled in the analyses performed for the subject retaining structure are 240 psf for wall heights of 25.0 feet, 26.5 feet and 37.0 feet. The surcharge load was modeled at 267 psf and 348 psf for wall heights of 18.5 feet and 13.0 feet, respectively. For the purposes of modeling the cantilever wall, the stem and footing thickness were estimated to be two feet and the length of the toe was estimated to be 10 percent of the wall height (0.1H). Table 17 summarizes the results of the CIP wall analyses performed for the subject retaining structure.

Table 17. Summary of CIP Wall Analyses

Station Interval*	Max Wall Height	Base Width	$CDR_{Sliding}$	Eccentricity	Meyerhof Uniform Pressure
39+00 to 40+00	37.0 ft	29.6 ft = 0.8H	1.12	3.13 ft = 0.12B	6,580 psf
40+00 to 40+50	25.0 ft	20.0 ft = 0.8 H	1.08	2.67 ft = 0.13B	4,670 psf
40+50 to 41+00	18.5 ft	14.8 ft = 0.8 H	1.01	1.92 ft = 0.16B	3,680 psf
41+00 to 41+50	13.0 ft	13.0 ft = 1.0 H	1.08	1.64 ft = 0.13B	2,740 psf
41+50 to 42+08	26.5 ft	21.2 ft = 0.8H	1.09	2.74 ft = 0.13B	4,910 psf

* Ramp 2 Stationing

FMSM initially estimated the base width modeled in the CIP wall analyses to be two-thirds of the wall height (2/3H). However, the base width was increased to 80 percent of the wall height (0.8H) to provide adequate resistance for sliding for walls greater than 19.0 feet in height. Walls less than 18.5 feet in height required the base width to be 90 percent of the wall height (0.9H), with walls less than 14.0 feet requiring the base width to be 100 percent of the wall height (1.0H).

9.3. Bearing Capacity Analyses of the Existing Soils

FMSM estimated the bearing capacity of the existing soils based on the same methods used for the MSE wall. A review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on fine-grained (clayey) alluvial soils between Stations 39+00 and 40+00 and shale (shot rock) embankment materials between Stations 40+00 and 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clayey materials from Stations 39+00 to 40+00. Cohesion values of 790 psf and 1,090 psf derived from unconfined compression test results and correlations of corrected SP N-values yields a nominal bearing capacity on the order of 4,265 psf for the clay alluvium. Since the remainder of the wall will be founded on the existing shale (shot rock) embankment the bearing capacity will be controlled by the friction angle of the shale embankment. A friction angle of 35 degrees was derived from correlations of the corrected SP N-values and yields nominal bearing capacities from 28,990 psf to 47,380 psf.

The resistance factors for bearing capacity outlined in Table 10.5.5.2.2-1 of the AASHTO LRFD Bridge Design Specifications were calibrated for rigid footings and range from 0.45 to 0.55. The KYTC Geotechnical Manual recommends a Factor of Safety (ASD methodology) of 2.0 to 3.0 for determination of allowable bearing capacity based on the amount and quality of strength data available. The KYTC typically recommends a factor of safety of 2.5, which Section C10.5.5.2.2 of the AASHTO LRFD Bridge Design Specifications indicates corresponds to a resistance factor of 0.55. Using a resistance factor of 0.55, the factored bearing capacity for the clay alluvium is on the order of 2,345 psf.

As with the MSE wall, the nominal bearing capacity was adjusted to incorporate an increase (or decrease) as applicable in bearing capacity for over excavation and replacement and for two-layered soil systems. Review of the planned bearing elevation and subsurface profile developed based on the drilling program indicate the wall will bear on existing shale (shot rock) embankment materials between Stations 40+00 and 42+08. However, the bearing elevation for the base of the wall is positioned near the weaker clay foundation soils so a reduction in the bearing capacity of the embankment materials will be necessary. Using the method for a two-layer soil system outlined in Section 10.6.3.1.2d of the AASHTO Specifications and a resistance factor (φ_b) of 0.55, FMSM developed Table 18 which outlines the factored bearing capacity applicable for various steps in retaining wall.

Table 18. Factored Bearing Capacity for CIP Wall Option

Station Interval**	Factored Bearing Capacity (q_R)* (psf)
40+00 to 40+50	4,330
40+50 to 41+00	6,150
41+00 to 41+50	11,260
41+50 to 42+08	3,400

* Using a Resistance Factor (φ_b) of 0.55.

** Ramp 2 Stationing

A review of the Meyerhof uniform pressure/required bearing capacity values determined for the CIP wall option as presented in Table 17 indicates the applied bearing pressures are greater than the factored bearing capacity for the clay alluvium between Stations 39+00 and 40+00 and for the shale (shot rock) embankment between Station 40+00 to 40+50 and Stations 41+50 and 42+08. As such, construction of the CIP wall option bearing directly on the in situ soils within the intervals mentioned above without some type of foundation soil modification will likely experience bearing capacity failure. FMSM recommends excavation of the foundation materials and replacement with Granular Embankment (crushed stone) as a means of spreading out the load exerted by the wall over a larger area, and thereby reducing the soil contact pressures to acceptable values. The estimated excavated area requiring granular embankment will vary as shown in Table 19 below. In addition, the interval between Stations 39+00 and 40+00 and Stations 41+50 and 42+08 will also require additional horizontal over excavation five feet beyond the wall perimeter and the use of geogrid reinforcement to assist in transferring wall loads to the foundation soil. Table 19 provides a summary of the over excavation and replacement requirements based on the anticipated wall loading and factored bearing capacity of the clay foundation soils.

Table 19. Summary of Over Excavation and Replacement for CIP Wall Option

Station Interval*	Maximum Wall Height	Approximate Bearing Elevation	Over Excavation Vertical	Over Excavation Horizontal
39+00 to 40+00	37.0 ft	459.0 ft	8 ft (451.0 ft)	8 ft**
40+00 to 40+50	25.0 ft	469.5 ft	2 ft (467.5 ft)	N/A
40+50 to 41+00	18.5 ft	475.5 ft	NA	NA
41+00 to 41+50	13.0 ft	480.5 ft	NA	NA
41+50 to 42+08	26.5 ft	467.5 ft	3 ft (464.5 ft)	5 ft**

* Ramp 2 Stationing
** Requires the use of geogrid

Section 11 of this report provides recommendations further outlining specific details and locations of foundation soil modifications.

9.4. Settlement Analyses

FMSM performed settlement analyses at select locations in order to develop an estimated settlement profile along the wall alignment similar to the methods used for the MSE walls. Based on the planned bearing elevations and over excavation depths previously discussed, it appears that the wall will bear on both shale (shot rock) embankment materials and alluvial clay foundation soils. The estimated settlement parameters derived for each soil horizon are presented on the Subsurface Data Sheets presented in Appendix D.

The applied pressures used in the analyses were based on the LRFD Service I load combinations and the resulting Meyerhof uniform pressure distribution beneath the wall using previously discussed traffic surcharge load. The results of the analyses indicate the potential for up to approximately 7.3 inches of settlement of the soils beneath the CIP wall. Table 20 presents a summary of the settlement analyses performed for the subject CIP option.

Table 20. Summary of Settlement Calculations for the CIP Option

Settlement Point Location*	Total Settlement (in.)	Total Settlement (ft)	Differential Settlement (ft)	Estimated Distance Over Which Settlement Occurs (ft)	Ratio of Differential Settlement
39+00	3.0	0.252			
			0.129	100	1 / 775
40+00	4.6	0.381			
			0.213	5	1 / 23
40+00	7.1	0.594			
			0.012	50	1 / 4,167
40+50	7.3	0.606			
			0.261	5	1 / 19
40+50	4.1	0.345			
			0.002	50	1 / 25,000

Table 20. Summary of Settlement Calculations for the CIP Option

Settlement Point Location*	Total Settlement (in.)	(ft)	Differential Settlement (ft)	Estimated Distance Over Which Settlement Occurs (ft)	Ratio of Differential Settlement
41+00	4.1	0.343			
			0.114	5	1 / 44
41+00	2.7	0.229			
			0.037	50	1 / 1,351
41+50	2.4	0.198			
			0.352	5	1 / 14
41+50	6.6	0.550			
			0.131	58	1 / 442
42+08	5.0	0.419			

* Ramp 2 Stationing

Settlement was generally estimated at “step” locations/changes in excavation depths. For the purpose of calculating a ratio of differential settlement, it was estimated that the differential settlement would occur over a distance of five feet at the step locations, otherwise the distance between settlement points was used.

Section C.11.6.2.2 of the AASHTO LRFD Bridge Design Specifications indicates that differential settlements on the order of 1 in 500 to 1 in 1,000 may overstress a reinforced concrete retaining wall. The differential settlements calculated for the CIP option vary from 1 in 442 to 1 in 25,000 between steps in the footing, with two of the five steps exhibiting differential settlements greater than 1 in 1,000. Section 11.6.1.1 of the AASHTO Specifications further indicates that rigid gravity walls should be supported by deep foundation elements when the foundation materials are prone to excessive total or differential settlement. The settlements calculated for the CIP option vary from a low of 2.4 inches to a high of 7.3 inches. Based on AASHTO LRFD specifications limiting total settlement of a full height MSE panel to two inches in magnitude, it is reasonable to estimate that two inches of total settlement for a CIP wall is also considered excessive. Therefore, based on total and differential settlement concerns, FMSM recommends that the CIP wall option be supported by deep foundation elements.

9.5. Deep Foundation Analyses

9.5.1. General

The subject retaining wall is adjacent to a bridge structure that will be widened and reconstructed as part of the Kennedy Interchange Project. The geotechnical consultants for the project are providing driven steel H-pile and concrete drilled shaft deep foundation options for the new substructure elements associated with this bridge. It is estimated that the driven H-pile options will be chosen for the final design. Therefore, only recommendations for driven piles are being provided for support of the CIP wall. If drilled shafts are chosen for final design, the Design Team should notify FMSM so that recommendations for drilled shafts may be provided.

9.5.2. Pile Capacity

Based on the results of the CIP settlement analyses, deep foundation elements bearing in the sand horizons overlying bedrock will be required and will rely primarily on friction resistance for axial capacity. A geotechnical engineer performed axial capacity estimates for three different H-pile sizes (12x53, 14x73 and 14x89). FMSM utilized the procedures outlined in the Federal Highway Administration Publication No. FHWA-HI-97-013, "Design and Construction of Driven Pile Foundations", and the computer program DRIVEN version 1.2, developed by Blue-Six Software, Inc. in conjunction with the FHWA, to estimate axial capacities of driven piles. The axial capacity calculations utilize soil parameters derived from the results of the field explorations and published correlations relating SP N-values to shear strengths. Appendix H provides Idealized Soil Profiles that outline the recommended soil parameters for use in lateral load analyses. Refer to Appendix I for single pile/shaft nominal axial capacity estimates for the S9280 (W65-10) retaining wall.

Selection of the resistance factors account for the type of loading (axial compression versus uplift) and the variability and reliability of models or methodologies used to determine nominal resistance (R_n) capacities. As mentioned previously, FMSM used the DRIVEN 1.2 computer program to perform the load capacity calculations for the subject bridge widening. Table 21 summarizes the applicable analysis methodologies utilized in the DRIVEN software as well as the resistance factors recommended by the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

Table 21. LRFD Resistance Factors for Driven Pile Capacity

Loading Condition	Resistance Mechanism	Analysis Methodology	Resistance Factor* (**φ**)
Nominal Resistance of Single Pile in Axial Compression – Static Analysis	Skin Friction and End Bearing – Clay and Mixed Soils	α-Method	0.35
	Skin Friction and End Bearing – Sand	Nordlund/Thurman Method	0.45
Uplift Resistance of Single Piles – Static Analysis	Side Resistance in Clay	α-Method	0.25
	Side Resistance in Sand	Nordlund Method	0.35

* From AASHTO LRFD Bridge Design Specifications, Fourth Edition portion of Table 10.5.5.2.3-1

Design of the foundation system will be based on the anticipated structural loads applied by the wall, which include the weight of the backfill as well as overturning forces from lateral earth pressure. Table 22 summarizes the estimated depths below the anticipated pile cap at which the proposed H-piles should extend to achieve the maximum total factored geotechnical axial resistance (TFGAR), based on static analysis and the resistance factors for driven piles presented in Table 21, above. The KYTC Geotechnical Branch recommends that the maximum TFGAR for each pile size be limited to the values presented in Table 22.

In accordance with Section 10.7.3.7 of the AASHTO LRFD Bridge Design Specifications, the pile lengths outlined in Table 22 were estimated by considering only the positive side friction and end bearing resistance below the zone contributing to downdrag.

Table 22. Summary of Driven Pile Capacities

Maximum Total Factored Geotechnical Axial Resistance[a] (tons)	Depth[b] (ft)	Tip Elevation (ft)	Total Factored Geotechnical Uplift Resistance[c] (tons)
12x53 H-pile			
100	70.0	398.1	82.9
14x73 H-pile			
140	71.5	396.6	116.3
14x89 H-pile			
170	74.5	393.6	139.4

[a] Excludes any positive resistance within downdrag zone
[b] Depth as measured from the bottom of the pile cap at 468.1 feet.
[c] Reported uplift resistance is for the corresponding pile length

The Designer should note that these estimates are for the maximum TFGAR listed in Table 22. The length estimates are based on the pile capacities presented in Table 22 and the length of pile subjected to downdrag. Should more or less capacity be required, the Designer should consult FMSM because the downdrag load and length of pile subjected to downdrag are a function of the pile length. Additionally, should the elevation of the bottom of the pile cap change, pile lengths and elevations presented in Table 22 would no longer be valid and should be adjusted accordingly.

The pile lengths outlined in Table 22 are based on static analysis and the corresponding resistance factors outlined in Table 21. If construction specifications require dynamic analysis during pile installation as outlined in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition, the Designer may estimate pile lengths for bid documents on the appropriate resistance factor outlined in the AASHTO specifications, based on the level of field testing and construction control. The pile capacity tables in Appendix I also include a column of factored capacities utilizing a resistance factor (ϕ_{dyn}) of 0.65, which corresponds to a specific level of dynamic analysis testing during pile installation.

9.5.3. Hammer Energy

FMSM performed static pile analyses to estimate the ultimate driving resistance that 12-inch or 14-inch steel H-piles will experience during the installation process for the proposed retaining wall. The engineering staff performed driveability analyses based on the bearing elevation and subsurface profile for the CIP retaining wall. FMSM utilized the guidelines presented in the FHWA publication "Soils and Foundations Workshop Manual" for the analyses.

The soil column contributing to driving resistance along the wall alignment includes clayey embankment materials, alluvial clay foundation soils, and the underlying sand and gravel layers. The analyses are based on steel H-piles being driven to the maximum depths shown in Table 22 for each of the three (3) pile types. Results of FHWA research and other literature regarding pile installation indicate that significant reductions in skin resistances occur during pile driving, primarily due to the dynamics of the installation process. Soils are remolded and pore water pressures apparently increase, causing reductions in shear strengths. The Kentucky Transportation Cabinet (KYTC) suggests the following reductions to skin resistances when estimating driving resistances:

Clay - 50%

Sands - 25%

FMSM estimated the driving resistances under the condition that no interruptions, and therefore no pile "set" characteristics would be experienced during the driving process. Drivability analyses were conducted using the GRLWEAP (Version 2005) computer program for 12x53, 14x73 and 14x89 steel H-piles using common hammer manufactures presented in the hammer database of the GRLWEAP program. Refer to Table 23 for approximate hammer energies to drive the various piles.

Table 23. Maximum Driving Depth for Hammer Energies

Approximate Hammer Energy (ft-kips)	Depth[a] (ft)	Tip Elevation[b] (ft)
12x53 H-pile		
23	70.0[c]	398.1[c]
40	70.0[c]	398.1[c]
60	70.0[c]	398.1[c]
14x73 H-pile		
23	65.5	402.6
40	71.5[c]	396.6[c]
60	71.5[c]	396.6[c]
14x89 H-pile		
23	65.0	403.1
40	73.5	394.6
60	74.5[c]	393.6[c]

[a] Depth as measured from the bottom of the pile cap
[b] Based on the estimate bottom of the pile cap at elevation 468.1 feet
[c] Depth/Elevation corresponding to the maximum TFGAR

The GRLWEAP analyses indicate that the ICE 60-S pile hammer, which imparts approximately 60 ft kips of energy, can drive the aforementioned piles to the maximum total factored geotechnical axial resistance without developing damaging compressive or tensile stresses within the pile, and without resulting in an excessive number of hammer blows per foot of driving. The FHWA publication titled "Soils and Foundations Workshop Manual-Second Edition" defines a reasonable range of hammer blows to be between 30 and 144 blows per foot for a steel H-pile. Upon selecting the pile size and length required to support the applied loads, the Designer should select the minimum hammer energy required to drive the piles to the specified depths listed in Table 23. Appendix J presents tables for H-pile driving resistance for the various pile sizes based on the soil profile at the structure site. The Designer may use Appendix J in conjunction with Appendix I to determine a minimum driving resistance required to drive the pile to a sufficient depth to achieve the specified capacity.

9.5.4. Downdrag Estimates

As discussed in section 9.4 of this report, FMSM is recommending that the foundation systems for the CIP wall option consist of deep foundation elements bearing in the sand horizons above the underlying bedrock. Fill placement behind the heel of the wall to accommodate the planned embankment widening will result in settlement of the foundation soils beneath the wall footprint. Settlement of these materials adjacent to the deep foundation elements will induce negative skin friction forces and apply downdrag loads to the piles. FMSM performed settlement calculations to estimate the magnitude of settlement of the soils beneath the wall in order to quantify the resulting downdrag forces. It should be noted that this settlement is a result of construction of the embankment behind the retaining wall and not a result of bearing pressures applied by the wall. Studies indicate that as little as 0.1 to 0.5 inches (3 to 12 mm) of settlement is sufficient to mobilize negative skin friction forces at the shaft/pile-soil interface.

FMSM performed calculations to estimate downdrag loads resulting from settlement of the foundation soils in relation to the planned deep foundation elements. As recommended by the AASHTO LRFD Bridge Design Specifications, the downdrag analyses are based on relative soil movements of 0.4 inches between the foundation elements and the surrounding soil mass. The calculations are based on the lengths outlined in Table 22 for the maximum total factored geotechnical axial resistance of the piles. If the wall design requires different lengths or capacities, the Designer should contact FMSM to re-evaluate the downdrag loads on the foundation elements. The calculations are based upon methods outlined in FHWA-HI-97-013 and FHWA-IF-99-025, which utilize soil strengths and effective stresses within the soil horizons. Table 24 outlines the potential negative skin friction estimates for driven pile deep foundation elements.

Table 24. Estimated Maximum Downdrag Loads for Driven Piles

Foundation Element Type	Total Factored Geotechnical Axial Resistance (tons)	Estimated Tip Elevation[a] (ft)	Estimated Element Length Subjected to Downdrag[b] (ft)	Estimated Maximum Downdrag Load	
				(kips)	(tons)
12x53 Steel H-Pile	100	398.1	21.2	47.6	23.8
14x73 Steel H-Pile	140	396.6	21.2	57.2	28.6
14x89 Steel H-Pile	170	393.6	22.6	66.0	33.0

[a] As outlined in Table 22
[b] As measured downward from the bottom of the pile cap

Because of the anticipated construction sequencing, downdrag/negative skin friction forces should be considered in the design of the foundation elements.

9.6. Lateral Squeeze

Studies conducted by the FHWA have shown that some bridge end bents supported on piles driven through thick deposits of compressible soils have tilted or rotated toward the embankment. The condition causing the structural deformation is the unbalanced fill loading on the area surrounding the end bents, which causes the foundation soils to move (squeeze) laterally. This unbalanced loading condition can be applied to that of a retaining wall bearing on driven piles. This squeeze can transmit a large lateral thrust along the length of the piles embedded within the compressible foundation soils, resulting in the tops of the piles rotating towards the embankment.

FHWA guidelines suggest that if the pressure exerted by the weight of the embankment exceeds three times the undrained shear strength of the foundation soils, the potential for lateral squeeze exists. Based on the subsurface exploration program, the cohesive soil horizons extend along the entire length of the retaining wall. A design value of 790 psf was derived for this alluvial clay layer, from the test data obtained for this wall. The pressure increase at the middle of the alluvial clay layer resulting from wall loading is approximately 3,441 psf between Stations 39+00 and 40+00 using the Service I load combination. Based on the noted criteria, the pressure applied by the wall does exceed three times the undrained shear strength of the clay foundation soils (3C = 3 x 790 = 2,370 psf) indicating that the potential for lateral squeeze exists for the CIP wall option and should be considered in the design of the wall foundation system. The FHWA "Soils and Foundation Workshop Manual" suggests that the anticipated lateral movement may be estimated as 25 percent of the fill settlement. A settlement analysis was conducted at Station 39+00. The analysis yielded an estimated settlement of 11.4 inches. Thus, the lateral deformation of the wall is estimated to be on the order of 2.9 inches. For the remainder of the retaining wall between Station 40+00 and 42+08 the bearing pressure increase based on the Service I load combination at the middle of the clay layer is less than three times the undrained strength of the clay foundation soils so the potential for lateral squeeze is low.

9.7. Settlement Below Deep Foundation Elements

Widening of the existing interstate and ramp embankments will result in settlement of the foundation soils underlying the planned retaining structure. Based on the anticipated construction sequencing (installation of deep foundation elements along the wall alignment, construction of the planned retaining wall, then construction of the embankment) the Designer should be aware that settlement will occur in the sand foundation soils below the tip elevation of the deep foundation elements. Settlement of the sands beneath the foundation elements will result in settlement of the pile foundation. It should be noted that this settlement is a result of construction of the embankment behind the retaining wall and not a result of structural loads placed on the pile foundation. Based on settlement calculations performed for the CIP retaining wall option and length estimates for the deep foundation elements, FMSM estimates this settlement to be less than ½-inch. Because of the cohesionless nature of the soils beneath the tip elevation of the deep foundation elements, this settlement should occur during construction of the embankment. The Contractor should be prepared to accommodate this settlement during construction.

9.8. Global Slope Stability

FMSM evaluated the global stability of the anticipated roadway embankment/CIP wall configuration utilizing the REAME 2004 slope stability program. Short-term analyses using total-stress shear-strength parameters for the foundation and embankment materials simulate conditions that will exist immediately following the construction of the embankment. Long-term analyses, using effective-stress shear-strength parameters, simulate conditions that will exist long after the embankment is constructed and excess pore pressures within the materials have dissipated. Table 25 presents a summary of the slope stability analyses performed for the CIP wall option.

Table 25. Summary of Global Slope Stability Analyses for CIP Option

Location	Global Slope Stability	
	Short Term	Long Term
Ramp 2 Station 39+00, right of centerline	1.3	2.4

FMSM evaluated global stability in terms of traditional ASD methodology using factors of safety. The KYTC Geotechnical Manual recommends minimum target factors of safety of 1.2 and 1.6 for short- and long-term global slope stability analyses, respectively, performed at structure locations. Based on a comparison of the KYTC minimum target factors of safety and the results of the global stability analyses summarized in Table 25, the calculated factors of safety exceed the recommended minimums. Subsurface Data Sheet 4 of 4 in Appendix D presents results of the slope stability analyses, including predicted minimum factors of safety, predicted failure surfaces, and modeled groundwater table positions.

10. Toe Wall Analyses

10.1. General

The plan view and cross-sections for the S9280 (W65-10) retaining wall show a toe wall is to be constructed approximately 20 to 50 feet to the east of the proposed face of the main retaining wall. The cross-sections indicate the wall will consist of a gravity-type, non-reinforced concrete structure measuring approximately six feet in height. Engineering analyses were performed to estimate bearing capacity and evaluate retaining wall and slope stability. These analyses are discussed further in the following sections.

10.2. Retaining Wall Analyses

This section of the report summarizes stability analyses performed for the planned toe wall. The retaining wall analyses were performed using spreadsheets developed by FMSM. The external stability of the proposed retaining wall was evaluated based on a maximum wall height of six feet and backfill consisting of both random backfill (common excavation) and granular material. The analyses are based on a functioning drainage system and do not account for hydrostatic pressures behind the wall.

The initial wall geometry evaluated was based on a gravity-type non-reinforced concrete structure conforming to KYTC Standard Drawing RGX 002-07. The external stability of the retaining wall was evaluated based on the following parameters.

Material Parameters Modeled in Gravity-Type Retaining Wall Stability Analyses

Parameter	Value used in Analyses
Wall Geometry	
Height	6 ft
Base Width	3 ft
Angle of Internal Friction for Material Behind the Wall:	
Random Backfill (Common Excavation)	27°
Granular Backfill	38°
Friction Angle Between Backfill and Back of Wall	
Random Backfill (Common Excavation)	17°
Granular Backfill	29°
Friction Angle Between Concrete and Foundation Material	
Lean Clay Foundation Soils	17°
Over Excavation & Replacement with Granular Embankment	29°
Unit Weight of Backfill Material (both common and granular)	120 pcf

The results of the stability analyses indicate that the standard gravity wall geometry will only work with over excavation of the foundation material and replacement with granular embankment and the fill behind the wall is restricted to granular embankment. Refer to Table 26 for a summary of the stability analyses performed for the gravity-type structure.

Table 26. Summary of Gravity-Type Retaining Wall Analyses

Backfill	Angle of Internal Friction (ϕ)	Max Wall Height	Base Width	$CDR_{Sliding}$	Eccentricity	Meyerhof Uniform Pressure
Lean Clay Foundation Soils						
Random Backfill	27°	6.0 ft	3.0 ft = 0.5H	0.31	1.07 ft = 0.36B	4,150 psf
Granular backfill	38°	6.0 ft	3.0 ft = 0.5 H	0.59	0.32 ft = 0.11B	1,785 psf
Granular Embankment Foundation						
Random Backfill	27°	6.0 ft	3.0 ft = 0.5 H	0.54	1.07 ft = 0.34B	4,150 psf
Granular backfill	38°	6.0 ft	3.0 ft = 0.5 H	1.01	0.32 ft = 0.11B	1,785 psf
Granular Embankment Foundation with Increased Base Width						
Random Backfill	27°	6.0 ft	7.2 ft = 1.2H	1.02	1.50 ft = 0.21B	2,530 psf

The results of the stability analyses summarized in Table 26 suggests that granular backfill will be required behind the wall and over excavation of foundation soils and replacement with granular embankment will be required to meet the minimum evaluation factors for the $CDR_{Sliding}$ and eccentricity. In addition, random backfill can be used but it requires over excavation of the foundation soils and an increased base width of the gravity wall. Based on the results of the stability analyses for the gravity-type wall, a cantilever-type wall geometry was also evaluated using the following parameters in order to provide a second option for the wall system.

Material Parameters Modeled in Cantilever-Type Retaining Wall Stability Analyses

Parameter	Value used in Analyses
Wall Geometry	
Height	6 ft
Base Width	4 ft
Stem Thickness	1 ft
Footing Thickness	2 ft
Angle of Internal Friction for Material Behind the Wall:	
Random Backfill (Common Excavation)	27°
Granular Backfill	38°
Friction Angle Between Backfill and Back of Wall	
Random Backfill (Common Excavation)	17°
Granular Backfill	29°
Friction Angle Between Concrete and Foundation Material	
Random Backfill (Common Excavation)	17°
Granular Backfill	29°
Unit Weight of Backfill Material (both common and granular)	120 pcf

Refer to Table 27 for a summary of the stability analyses performed for the cantilever-type structure.

Table 27. Summary of Cantilever-Type Retaining Wall Analyses

Backfill	Angle of Internal Friction (ϕ)	Max Wall Height	Base Width	$CDR_{Sliding}$	Eccentricity	Meyerhof Uniform Pressure
Lean Clay Foundation Soils						
Random Backfill	27°	6.0 ft	4.0 ft = 0.7H	0.24	0.27 ft = 0.07B	2,000 psf
Granular backfill	38°	6.0 ft	4.0 ft = 0.7H	0.54	0.04 ft = 0.01B	1,310 psf
Granular Embankment Foundation						
Random Backfill	27°	6.0 ft	4.0 ft = 0.7H	0.41	0.27 ft = 0.07B	2,000 psf
Granular backfill	38°	6.0 ft	4.0 ft = 0.7H	0.91	0.04 ft = 0.01B	1,310 psf
Granular Embankment Foundation with Increased Base Width						
Granular Backfill	38°	6.0 ft	4.8 ft = 0.8H	1.00	0.10 ft = 0.02B	1,275 psf

The retaining wall analyses for the cantilever-type wall also indicate that over excavation of the foundation soils and backfill with granular embankment with an increase in the base width of the wall will be required to meet the minimum evaluation factors for the $CDR_{Sliding}$. Based on a review of the results presented in Table 27, it is recommended that random backfill be excluded from use behind the toe wall for the cantilever-type retaining wall. Recommendations are being provided on the placement of granular backfill.

10.3. Bearing Capacity Analyses of the Existing Soils

Based upon the information derived from drilling, sampling, and lab testing operations conducted along the planned retaining wall system, ultimate bearing capacity estimates were performed for comparison with the induced wall loadings. The methodology used to calculate the nominal bearing capacity (q_n) is presented in the AASHTO LRFD Bridge Design Specifications, Fourth Edition Section 10.6.3 and the US Army Corps of Engineers "Bearing Capacity of Soils", EM 1110-1-1905.

Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicates the wall will be founded on clayey soils. Thus, the bearing capacity will be controlled by the short-term strength of the clays. Because of the limited amount of testing performed for the subject wall, the bearing capacity analyses utilized the results of laboratory testing performed for the larger companion retaining wall to the west. A value of cohesion of 790 psf derived from unconfined compression test results and correlations of corrected SP N-values yields a recommended allowable bearing capacity on the order of 2,230 psf using a width of wall of 3.0 feet and 4.8 feet for the gravity wall and the cantilever wall, respectively. This value will be compared against the applied bearing pressures estimated as part of retaining wall analyses discussed in Section 10.2 above.

11. Conclusions and Recommendations

FMSM developed the following recommendations based upon reviews of available data, information obtained during the field exploration, results of laboratory testing and engineering analyses, and discussions with the Designer and KYTC personnel. The recommendations are specific to the wall geometries, heights, and bearing elevations discussed herein, derived from structure drawings downloaded from the KTA ProjectWise website on March 1, 2007. If roadway design modifications result in retaining wall geometries different than those discussed and evaluated herein, FMSM should be notified and provided with the design changes to re-evaluate the planned retaining wall system and modify the recommendations as applicable.

11.1. General

11.1.1. Design of the subject retaining structure should be in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

11.1.2. Based on the subsurface conditions encountered in borings performed along the wall alignment, the wall will be a soil bearing structure. It is recommended that the minimum wall embedment depth be in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

11.1.3. Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on clayey soils from Station 39+00 to 40+00 and shale (shot rock) embankment from Station 40+00 to 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clays and the friction angle of the shale. Based on a value of cohesion of 790 psf and a friction angle of 35 degrees for the alluvial clays and shale (shot rock) embankment materials, respectively, derived from unconfined compression test results and correlations of corrected SP N-values, the estimated nominal bearing capacity (q_n) of the soils between Station 39+00 and 40+00 is on the order of 4,226 psf. The estimated nominal bearing capacity (q_n) of the shale embankment between Station 40+00 and 42+08 ranges from 27,050 psf to 44,308 psf.

11.1.4. Construction of the planned retaining wall will require excavations at the toe of the existing interstate and ramp embankments as well as temporary excavations within the embankments themselves. The Contractor should evaluate the stability of the existing embankments and adjacent structures in conjunction with temporary excavations to verify that the planned excavation/bracing/shoring system maintains the stability of the highway embankment.

11.1.5. Temporary wall slopes and foundation excavations in soil shall be properly designed, or should be properly braced/shored to reduce the possibility of collapse and provide adequate safety to people working in or around the excavations. Bracing/shoring shall be performed in accordance with applicable federal, state and local guidelines.

11.1.6. At the writing of this report, a borrow source for embankment material has not been identified. The engineering analyses performed for the retaining wall options are based on estimated soil strength parameters for the retained fill and embankment materials. It is recommended that borrow material to be used for embankment construction meet the following minimum strength parameters.

Embankment Material		Retained Fill	
Total Stress	Effective Stress	Total Stress	Effective Stress
c = 1400 psf ϕ = 0° γ = 120 pcf	$\bar{c}$ = 200 psf $\bar{\phi}$ = 23° γ = 120 pcf	c = 1400 psf ϕ = 0° γ = 120 pcf	$\bar{c}$ = 170 psf $\bar{\phi}$ = 27° γ = 120 pcf

The retained fill material shall be placed in the entire area between the wall and a 1:1 (H:V) line sloping upward and away from the base of the heel of the wall for a CIP wall or the base of the reinforced material for a MSE wall to the top of the wall. <u>Non-durable shales and fat clays (USCS classification of CH) should specifically be excluded from use within this zone</u>.

The Contractor shall perform laboratory testing to confirm that the minimum total stress and effective stress strength parameters are equal to or greater than the above values per material type for each borrow area. The test results shall be submitted to the Engineer for approval.

11.1.7. Fill materials associated with interstate construction and/or previous development in the City of Louisville were encountered within each of the five borings drilled along the wall alignment. Because the structure site is located within an area of previous site grading and construction, the Contractor should anticipate encountering fill materials along the wall alignment. The excavations shall be deepened as necessary to provide an adequate bearing medium.

11.1.8. Granular embankment used as backfill and/or for over excavation and replacement shall be non-erodible and shall conform to the requirements of Section 805 of the current Kentucky Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The granular embankment material shall be wrapped with Type IV geotextile fabric meeting the requirements of Section 843 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction to provide separation from the clay embankment and/or foundation materials.

11.1.9. Soils exposed within the bottoms of footing trenches shall be observed for suitability by a geotechnical engineer or an engineering technician working under his/her direct supervision. Old fill or unsuitable material which might be encountered shall be removed. Areas disturbed by the excavation process should be restored utilizing proper compaction methods.

11.1.10. If soft, wet soils are encountered in the foundation excavations, they should be undercut a minimum depth of two (2) feet and backfilled to the design bearing elevation using non-erodible Granular Embankment conforming to Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The bottoms of the foundation excavations should be proof-rolled to restore the in-place density of any soil disturbed in the excavation process. Isolated zones of loose or wet soil may also be stabilized using KY size No. 2, 3, or 23 stone, as specified in Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

11.2. Mechanically Stabilized Earth (MSE) Wall

It is FMSM's understanding that the MSE option for the subject retaining wall will exhibit the geometry summarized in the following table. The height of the wall is as measured from the base of the wall up to roadway grade. The backfill slope behind the wall will be level.

Summary of Wall Configuration Evaluated for MSE Option

Alignment	Station Limits	Maximum Wall Height	Base of Wall Elevation*
Ramp 2	39+00 to 40+00	36.0 ft	460.0 ft
Ramp 2	40+00 to 40+50	24.0 ft	470.5 ft
Ramp 2	40+50 to 41+00	17.5 ft	476.5 ft
Ramp 2	41+00 to 41+50	12.0 ft	481.5 ft
Ramp 2	41+50 to 42+08	26.5 ft	467.5 ft

* See Recommendation 11.2.6 for foundation soil modification

11.2.1. Based on the nominal bearing capacities (q_n) outlined in Recommendation 11.1.3 and a resistance factor (φ_b) of 0.67, the factored bearing capacity for the MSE wall option should be designed using the following factored bearing capacities (q_R):

Station Interval	Factored Bearing Capacity (q_R)*
Ramp 2 39+00 to 40+00	2,830 psf
Ramp 2 40+00 to 40+50	5,730 psf
Ramp 2 40+50 to 41+00	8,870 psf
Ramp 2 41+00 to 41+50	16,220 psf
Ramp 2 41+50 to 42+08	4,180 psf

* Applicable for no over excavation.

11.2.2. Based on eccentricity requirements outlined in Section 11.6.3.3 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition, it is recommended that the minimum reinforcing strap length used for design and construction of the MSE wall option conform to the guidelines outlined in the table below. However, the minimum strap length should not be less than eight (8) feet.

Guidelines For Minimum Reinforcement Strap Length for MSE Walls

Wall Height	Minimum Strap Length
29 ft ≤ H	0.7H
16 ft ≤ H < 29 ft	0.8H
12 ft ≤ H < 16 ft	0.9H
9 ft ≤ H < 12ft	1.0H

This strap length may need to be increased by the wall designer depending on the results of the internal wall stability analyses. The wall designer should verify MSE wall stability against sliding, eccentricity and bearing capacity failure, based on the final wall design dimensions. The stability analyses should be performed in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

11.2.3. The minimum wall embedment shall be as specified in Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition, or two (2) feet, whichever is greater, as measured from the ground surface in front of the wall down to the base of the wall.

11.2.4. The internal design of the MSE wall shall be performed in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition. The pullout resistance shall be based on a ϕ = 34°, unless specific laboratory tests are conducted to obtain pullout design parameters.

11.2.5. It is recommended that the gradation of the reinforced soil conform to guidelines presented in the FHWA publication "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" and Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. During MSE wall construction this material should be wrapped with an engineering fabric (Geotextile) for separation. The engineering fabric should conform to the requirements of Section 843 of the current Kentucky Standard Specifications for Road and Bridge Construction.

11.2.6. Based on the loading conditions calculated for the MSE wall, and the factored bearing capacity of the existing foundation soils, it is recommended that portions of the wall foundation area be over-excavated and backfilled with granular embankment in an effort to spread the wall load over a large area and meet bearing capacity requirements. The areas requiring foundation soil modification and the type of modification recommended are presented in the table below.

Recommended Foundation Soil Modification for MSE Wall

Station Interval*	Maximum Wall Height	Bearing Capacity at the base of the Over Excavation**	Recommended Foundation Soil Modification
39+00 to 40+00	36.0 ft	5,670 psf (Alluvial Clay)	Over-excavate to a minimum depth of 8 feet below the base of the MSE wall and 8 feet horizontally beyond the wall perimeter (assuming a 1H:1V backfill slope). The excavation should be backfilled with geogrid reinforced granular embankment. (Recommendation 11.1.8.)
40+00 to 40+50	24.0 ft	5,730 psf (Shale Embankment)	No soil modification required.
40+50 to 41+00	17.5 ft	8,870 psf (Shale Embankment)	No soil modification required.
41+00 to 41+50	12.0 ft	16,220 psf (Shale Embankment)	No soil modification required.
41+50 to 42+08	26.5 ft	4,160 psf (Alluvial Clay)	Over-excavate to a minimum depth of 5 feet below the base of the MSE wall and 5 feet horizontally beyond the wall perimeter (assuming a 1H:1V backfill slope). The excavation should be backfilled with geogrid reinforced granular embankment. (Recommendation 11.1.8.)

* Ramp 2 Stationing

** The values presented in the table include an increase in bearing capacity for over excavation and an increase or decrease for a two-layer system, as applicable.

11.2.7. Additional horizontal over-excavation of the foundation soils is recommended in select areas as outlined in Recommendation 11.2.6. to spread the applied wall loads over a larger area. Biaxial geogrid reinforcement shall be incorporated into the granular replacement of the over excavated foundation soils within these areas to assist load transfer. The geogrid reinforcement shall be placed between one foot layers of compacted non-erodible granular embankment. The biaxial geogrid shall extend the full width and length of the area requiring horizontal over excavation. At a minimum, the biaxial geogrid reinforcement shall exhibit the following Machine Direction (MD) values:

Index Property	MD Value*
2% Junction Tension Modulus in Use	18,200 lb/ft
Junction Strength in Use at 2% Strain	370 lb/ft

* In accordance with GRI-GG2-87

11.2.8. Based on settlement calculations performed along the wall alignment, settlements up to 8.7 inches can be expected. Differential settlements on the order of 4.7 inches occurring over a wall length of 5 feet (1/13) should be anticipated and considered in the design of the MSE wall. The wall designer should select the panels and size the joint widths between the wall panels to accommodate the anticipated settlements. It is FMSM?s understanding that the permanent fascia will not be installed until an appropriate amount of settlement has occurred as indicated in Recommendation 11.2.11. If this cannot be done, ground improvement techniques such as, additional over excavation and replacement, or use of stone columns may be warranted to reduce the anticipated settlement. The over-excavation limits recommended in Recommendation 11.2.6 are required to modify foundation conditions to achieve adequate bearing capacity. Additional over-excavation and replacement of soils with select embankment would be required to reduce the magnitude of settlement.

11.2.9. It is recommended that vertical slip joints be incorporated into the design and construction of the MSE wall at the following locations in conjunction with a “step" in the wall bearing elevation. It is recommended that vertical slip joints also be constructed at any additional “steps” introduced into the planned bearing elevation by the wall Designer.

Ramp 2 Station
40+00
40+50
41+00
41+50

11.2.10. If the placement of an obstruction in the wall reinforcement zone such as drainage structures, signal or sign foundations, guardrail posts, or the bridge foundation system (piles or drilled shafts/caissons) cannot be avoided, the design of the wall near the obstruction shall be modified using one of the following alternatives:

a. Place a structural frame (collar or yoke) around the obstruction which is capable of carrying the load from the reinforcement in front of the obstruction to the reinforcement connected to the structural frame behind the obstruction.

b. If the soil reinforcements consist of discrete strips or bar mats rather than continuous sheets, depending on the size of the obstruction, it may be possible to splay the reinforcements around the obstruction.

c. Reinforcement layers shall not be structurally connected to any foundation elements.

11.2.11. Settlement analyses were performed along the planned wall alignment in order to develop settlement profiles for the proposed structure. It is FMSM's understanding that the wall will be constructed and allowed to settle prior to the attachment of the permanent fascia. Time rate of settlement calculations indicate 90 percent of primary consolidation will occur in about 189 days (27 weeks). In order to monitor the settlement of the wall, settlement platforms shall be furnished and installed by the Contractor at the following approximate locations. Installation of the platforms shall be in accordance with Section 216 of the current Standard Specifications for Road and Bridge Construction and Standard Drawing RGX-015.

Ramp 2 Station
39+50, 10? Rt.
40+25, 10? Rt.
40+75, 10? Rt.
41+25, 10? Rt.
41+75, 10? Rt.

The exact locations shall be determined by the Engineer and a representative of the Division of Structural Design, Geotechnical Branch once wall layouts have been finalized. Settlement readings shall begin with an initial reading upon placement of the platform prior to beginning embankment/retaining wall construction. Readings shall continue periodically during and following completion of the subject roadway embankments and retaining structures. The settlement platforms shall be left in place for future readings after the project is completed. Necessary forms for recording settlement measurements can be obtained from the Geotechnical Branch at the request of the Engineer. The Engineer will be responsible for reading and recording the settlement data. The Geotechnical Branch will be responsible for evaluation of the actual settlement data. Installation of the permanent fascia shall not begin until the Geotechnical Branch has determined that an adequate percent of consolidation of the foundation soils has been achieved. The Contractor shall be responsible for replacing all damaged settlement platforms at no extra cost.

11.3. Cast-in-Place (CIP) Cantilever Retaining Wall

It is FMSM's understanding that the CIP option for the subject retaining wall will exhibit the geometry summarized in the following table. The height of the wall is as measured from the base of the wall up to roadway grade. The backfill slope behind the wall will be level.

Summary of Wall Configuration Evaluated for CIP Option

Alignment	Station Limits	Maximum Wall Height	Base of Wall Elevation
Ramp 2	39+00 to 40+00	37.0 ft	459.0 ft
Ramp 2	40+00 to 40+50	25.0 ft	469.5 ft
Ramp 2	40+50 to 41+00	18.5 ft	475.5 ft
Ramp 2	41+00 to 41+50	13.0 ft	480.5 ft
Ramp 2	41+50 to 42+08	26.5 ft	467.5 ft

11.3.1. The minimum wall embedment should be three (3) feet as measured from the ground surface in front of the wall to the base of the footing to provide approximately one (1) foot of soil cover over the wall footing.

11.3.2. Backfill behind the wall can consist of retained fill as defined in Recommendation 11.1.6. or non-erodible granular embankment as defined in Recommendation 11.1.8.. Coefficients of active earth pressure (K_a) were determined based on Coulomb earth pressure theory using phi angles of 27 and 38 degrees, a vertical back of wall, and friction angles between the back of the wall and backfill of 17 and 29 degrees. Based on a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

Slope of Backfill	Common Backfill (ϕ = 27°)		Granular Embankment (ϕ = 38°)	
	Coefficient of Active Earth Pressure (K_a)	Equivalent Fluid Pressure Per Linear Foot	Coefficient of Active Earth Pressure (K_a)	Equivalent Fluid Pressure Per Linear Foot
Level	0.335	40 psf	0.218	26 psf
3:1 (H:V)	0.464	56 psf	0.274	33 psf
2:1 (H:V)	0.714	86 psf	0.323	39 psf

Drainage systems consisting of free draining material and filter fabric shall be placed directly behind the wall and be a minimum thickness of two feet. Use of filter fabric will help reduce the infiltration of fines into the granular material behind the wall and help reduce clogging of the drainage system. In addition, weep holes should also be provided in the design of the walls. If a drainage system is not provided, the design should incorporate full hydrostatic forces behind the wall.

11.3.3. Based on total and differential settlement concerns, it is recommended that the CIP wall option be supported by deep foundation elements. The design of the deep foundation system should be based on the following toe and heel pressures calculated for the wall configurations described herein.

Applicable Bearing Pressures for Design of Deep Foundations for CIP Wall Option

Station Interval*	Wall Geometry**			Calculated Pressures		
	Maximum Height	Footing Width	Toe Width	Meyerhof Uniform	Maximum Toe	Minimum Heel
39+00 to 40+00	37.0 ft	0.8H	0.1H	4,880 psf	7,950 psf	3,520 psf
40+00 to 40+50	25.0 ft	0.8H	0.1H	4,670 psf	5,700 psf	2,330 psf
40+50 to 41+00	18.5 ft	0.8H	0.1H	3,680 psf	4,550 psf	1,680 psf
41+00 to 41+50	13.0 ft	1.0H	0.1H	2,740 psf	3,170 psf	1,830 psf
41+50 to 42+08	26.5 ft	0.8H	0.1H	4,910 psf	5,990 psf	2,480 psf

* Ramp 2 Stationing
** H = Wall Height

The bearing pressures provided above were determined based on the wall geometries outlined in the table. If the final design results in retaining wall geometries different than those outlined above, the retaining wall designer should perform analyses to determine the appropriate bearing pressures for design of the foundation system.

11.3.4. Axial capacity estimates for single steel H-piles are provided in Appendix I. The following table provides estimated pile lengths applicable for the recommended maximum total factored geotechnical axial resistances (TFGAR) along the wall alignment. Upon determination of the final H-pile locations, arrangement, and loads, the Designer should use the estimates to determine the H-pile size and length for each pile. However, the Designer should note that these estimates are for the TFGAR referenced in the following table only. Should more or less capacity be required, the Designer should consult FMSM because the downdrag load and length of pile subjected to downdrag are a function of the pile length.

Summary of Driven Pile Capacities

Maximum Total Factored Geotechnical Axial Resistance[a] (tons)	Depth[b] (ft)	Tip Elevation (ft)	Total Factored Geotechnical Uplift Resistance[c] (tons)
12x53 H-pile			
100	70.0	398.1	82.9
14x73 H-pile			
140	71.5	396.6	116.3
14x89 H-pile			
170	74.5	393.6	139.4

[a] Excludes any positive resistance within downdrag zone
[b] Depth as measured from the bottom of the pile cap.
[c] Reported uplift resistance is for the corresponding pile length

11.3.5. The TFGAR estimates provided in Appendix I were derived using the following resistance factors, as recommended by the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

Loading Condition	Resistance Mechanism	Analysis Methodology	Resistance Factor* (φ)
Nominal Resistance of Single Pile in Axial Compression – Static Analysis	Skin Friction and End Bearing – Clay and Mixed Soils	α-Method	0.35
	Skin Friction and End Bearing – Sand	Nordlund/Thurman Method	0.45
Uplift Resistance of Single Piles – Static Analysis	Side Resistance in Clay	α-Method	0.25
	Side Resistance in Sand	Nordlund Method	0.35

* From AASHTO LRFD Bridge Design Specifications, Fourth Edition, portion of Table 10.5.5.2.3-1

11.3.6. If load testing and/or dynamic analysis of driven piles in soil is conducted, the LRFD resistance factors used to determine the factored axial capacity for design purposes may be revised as outlined in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition based on site variability and the number and type of tests performed. If the Designer performed lateral capacity analyses based on the pile lengths outlined in Recommendation 11.3.4, the lateral capacity analyses will need to be revisited if the pile lengths are revised based on load testing and/or dynamic analysis.

11.3.7. Because the widening of the roadway embankment will be constructed after installation of the deep foundation elements recommended for support of the CIP wall, the piles will be subjected to downdrag loads resulting from settlement of the foundations soils. One of the following alternatives may be implemented to reduce the downdrag loads:

a. Coat piles with bitumen slip layer within the zone subjected to downdrag to allow movement between the soil and the piles. Current practice allows for as much as 90 percent reduction in downdrag forces with this method.

b. Predrill and provide a polypropylene or steel sleeve for the pile to reduce down-drag. This method only prevents contact between the pile and adjacent soils.

c. Design the embankment with lightweight fill to reduce the overall settlement of the foundation soils.

11.3.8. As noted, all pile capacities presented in Appendix I are for single piles. In addition to applying appropriate resistance factors, individual capacities for piles in group configurations may be further reduced depending upon soil type, bearing condition of the pile cap, or center-to-center spacing as recommended in the AASHTO LRFD Bridge Design Specifications, Fourth Edition. The following criteria should be observed:

CTC Spacing	Group Efficiency Factor		
	Cohesive Soils		Cohesionless Soils
	Cap not in firm Contact with Ground	Cap in firm Contact with Ground	Cap in or not in firm Contact with Ground
6B	1.00	1.00	1.00
2.5B	0.65	1.00	1.00

The notation "B" is the pile diameter and the percent reduction can be linearly interpolated between the values and spacing provided.

11.3.9. The AASHTO LRFD Bridge Design Specifications recommend a resistance factor for horizontal geotechnical resistance of a single pile or pile group of 1.0 for lateral capacity analyses. Appendix H provides Idealized Soil Profiles that outline the recommended soil parameters for use in lateral load analyses.

11.3.10. Use Grade 50 steel H-piles as friction piles. Piles should be driven to the target elevation and then left for a minimum of one day to allow for dissipation of excess pore pressures caused by the pile installation process. This should allow the soil to "set-up". After the one day waiting period, re-strike the piles to see if an adequate capacity has been achieved.

11.3.11. Hammer energies which could drive the pile section were based on the ultimate driving resistance that 12x53, 14x73 and 14x89 steel H-piles would experience during the installation process. The results of these calculations are presented in the following table:

Maximum Driving Depth for Hammer Energies

Approximate Hammer Energy (ft-kips)	Depth[a] (ft)	Tip Elevation[b] (ft)
12x53 H-pile		
23	70.0[c]	398.1[c]
40	70.0[c]	398.1[c]
60	70.0[c]	398.1[c]
14x73 H-pile		
23	65.5	402.6
40	71.5[c]	396.6[c]
60	71.5[c]	396.6[c]
14x89 H-pile		
23	65.0	403.1
40	73.5	394.6
60	74.5[c]	393.6[c]

[a] Depth as measured from the bottom of the pile cap
[b] Based on the estimate bottom of the pile cap at elevation 468.1 feet
[c] Depth/Elevation corresponding to the maximum TFGAR

11.3.12. Upon selecting the pile size and length required to support the applied loads, the Designer should select the minimum hammer energy required to drive the piles to the specified depths from the table presented in Recommendation 11.3.11 above. The Designer should place a note on the drawings that states: A hammer system capable of delivering a minimum energy of ___ foot-kips will be necessary to drive the piles without encountering excessive blow counts and over-stressing the piles. The Contractor should submit appropriate pile driving systems to the Kentucky Transportation Cabinet for approval prior to the installation of the first pile. Approval of the pile driving system by the Engineer will be subject to satisfactory field performance of the pile driving procedures.

11.3.13. Upon selecting the pile size and length required to support the applied loads, the Designer should select the minimum driving resistance required to install the pile to the design depth from the tables provided in Appendix J. This driving resistance should be reported to the Contractor to aid in determining when/if the pile has been driven to a sufficient depth to achieve the specified capacity.

11.3.14. Pile types, driving systems and installations should conform to current AASHTO LRFD Bridge Design Specifications unless otherwise specified.

11.3.15. Drivability studies were performed assuming continuous driving. If interruptions in driving individual piles should occur, difficulties in continuing the installation process will likely occur due to pile "set-up" characteristics.

11.3.16. The AASHTO LRFD Bridge Design Specifications, Fourth Edition recommends the following resistance factors for determining the structural capacity of steel H-piles.

Loading Condition	Resistance Factor*	
	Piles Subjected to Damage From Severe Driving Conditions**	Good Driving Conditions
Axial Resistance In Compression	$\varphi_c = 0.50$	$\varphi_c = 0.60$
Combined Axial and Flexural Resistance	N/A	$\varphi_c = 0.70$ $\varphi_f = 1.00$

* As specified in Section 6.5.4.2 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition

** Apply these values only to the section of the pile likely to be damaged during driving (Section 6.15.2 of the AASHTO Specifications)

11.4. Toe Wall

It is FMSM's understanding that the subject retaining wall will measure approximately 300 feet in length and exhibit a maximum height of approximately 6 feet, as measured from the base of the footing to the top of the wall. The backfill configuration behind the wall will be a 2H:1V slope.

11.4.1. Based on the depth to bedrock and the anticipated wall loading, it is recommended that the retaining wall be supported by a yielding foundation system. The allowable bearing capacity of the underlying soil material is 2,230 pounds per square foot.

11.4.2. To maintain an acceptable factor for the $CDR_{Sliding}$, it is recommended that a minimum of one foot of material be excavated below the wall footprint and backfilled with non-erodible granular embankment as defined in Recommendation 11.1.8.

11.4.3. The minimum wall embedment shall be two feet, as measured from the ground surface in front of the wall down to the base of the footing.

11.4.4. Retaining wall stability analyses indicate the geometry for a six foot tall standard gravity wall (KYTC Standard Drawing RGX-002-07) will meet the minimum factor for $CDR_{Sliding}$ based on the AASHTO LRFD Bridge Design Specifications, Fourth Edition. If a gravity type retaining wall is chosen, the foundation soils will need to be over excavated and replaced with granular embankment (see Recommendation 11.4.2.), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Recommendation 11.1.8. As another option, a gravity type retaining wall can be used with random backfill provided that the foundation soils are over excavated and replaced with granular embankment and the base width is widened to 7.2 feet.

Using a phi angle of 38 degrees, a wall height measuring 6 feet, a base width measuring 3 feet, an angle between back of the wall and vertical equal to 14.0 degrees, a friction angle between the back of the wall and the granular backfill equal to 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	41 psf
3:1(H:V)	53 psf
2:1(H:V)	64 psf

The backfill shall be placed in the entire area between the wall and a 1:1(H:V) line sloping upward away from the base of the heel of the wall to the top of the wall. Type IV geotextile fabric shall be placed on the 1:1(H:V) slope to reduce the infiltration of fines into the granular material behind the wall and help prevent clogging of the drainage system. The drainage system shall consist of 4 inch diameter pipe with weepholes installed at locations as indicated by KYTC Standard Drawing RGX 002-07 or by the Designer; and/or perforated pipe installed at the base of the wall and “daylighted” to promote “dewatering” of the granular backfill.

11.4.5. Retaining wall stability analyses indicate a cantilever-type retaining wall measuring six feet in height can meet the minimum factor for $CDR_{Sliding}$ and eccentricity provided that the foundation soils are over excavated and replaced with granular embankment (see Recommendation 11.4.2.). In addition, the footing width will need to be increased to 4.8 feet (0.8H), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Recommendation 11.1.8.

Using a phi angle of 38 degrees, a vertical back of wall, friction angle between the back of the wall and backfill of 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	26 psf
3:1(H:V)	33 psf
2:1(H:V)	39 psf

Drainage systems consisting of free draining material and filter fabric shall be placed directly behind the wall and be a minimum thickness of two (2) feet. Use of filter fabric will help reduce the infiltration of fines into the granular material behind the wall and help reduce clogging of the drainage system. In addition, weep holes shall also be provided in the design of the walls. If a drainage system is not provided, the design shall incorporate full hydrostatic forces behind the wall.

12. Closing

12.1. General soil and rock descriptions and indicated boundaries are based on an engineering interpretation of all available subsurface information and may not necessarily reflect the actual variation in subsurface conditions between borings and samples. Collected data and field interpretation of conditions encountered in individual borings are shown on the geotechnical drawings included in Appendixes C and D.

12.2. The observed water levels and/or conditions indicated on the boring logs are as recorded at the time of exploration. These water levels and/or conditions may vary considerably, with time, according to the prevailing climate, rainfall or other factors and are otherwise dependent on the duration of and methods used in the exploration program.

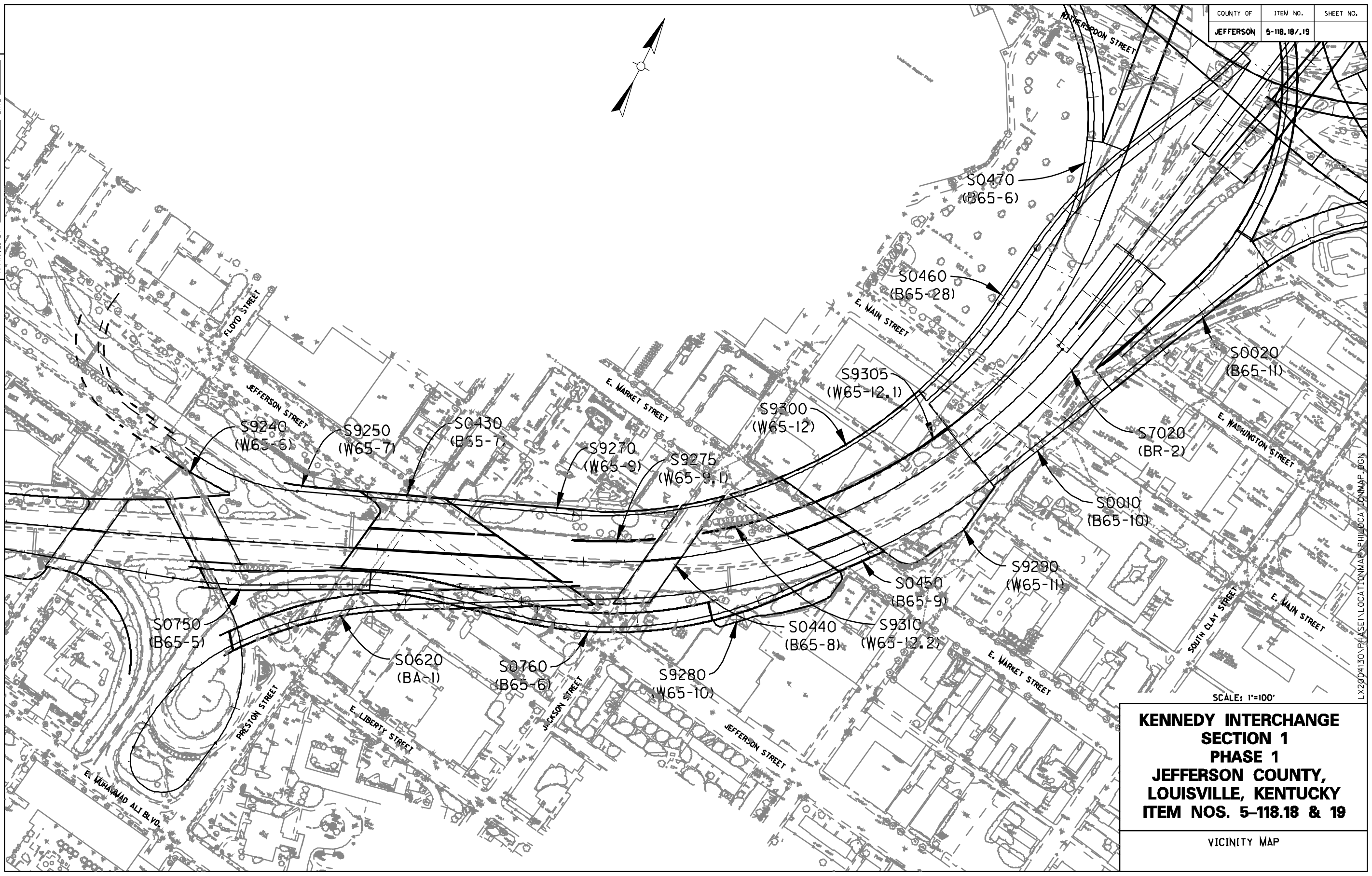
12.3. Sound engineering judgment was exercised in preparing the subsurface information presented herein. This information was prepared and is intended for design and estimating purposes. Its presentation on the plans or elsewhere is for the purpose of providing intended users with access to the same information available to the KYTC. This subsurface information interpretation is presented in good faith and is not intended as a substitute for personal investigations, independent interpretations or judgments of the Contractor.

12.4. All structure details shown herein are for illustrative purposes only and may not be indicative of the final design conditions shown in the contract plans.

12.5. The scope of services for this portion of the project did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below the site. Any statements in this report or on the soil boring logs regarding odors noted or unusual or suspicious conditions observed are for the information of the KYTC and should not be construed as an environmental evaluation.

Appendix A

Location Map



KENNEDY INTERCHANGE
SECTION 1
PHASE 1
JEFFERSON COUNTY,
LOUISVILLE, KENTUCKY
ITEM NOS. 5–118.18 & 19

VICINITY MAP

PREPARED BY ____ DATE ____
CHECKED BY ____ DATE ____
APPROVED BY ____ DATE ____

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DATE: $$$$DATE$$$$
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Appendix B

Client Drawings from ProjectWise

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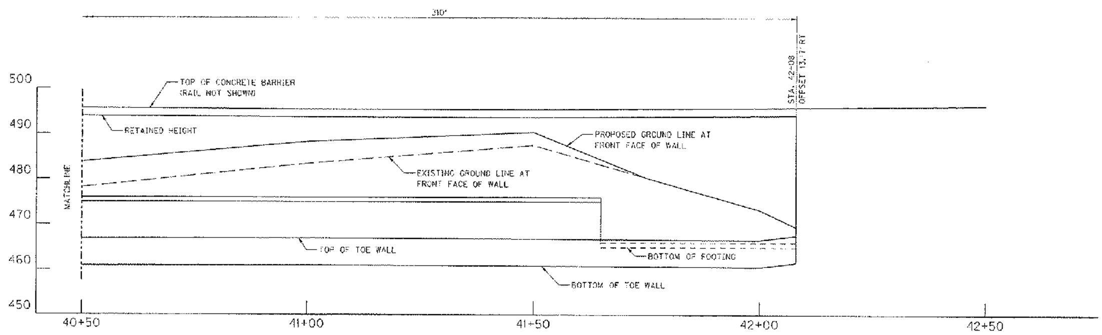

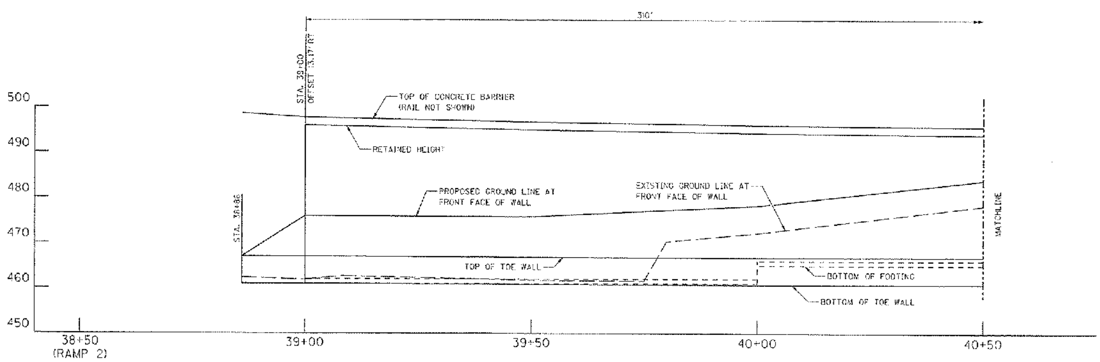

ELEVATION WALL W65-10

Total Wall Length: 310'
Toe Wall Length: 373'

59280_W65-10 ASF.dgn downloaded from Projectwise on 3/1/07

NOTE:
- OFFSET GIVEN TO FRONT FACE OF RETAINING WALL
- DIMENSIONS INDICATE LENGTH ALONG FRONT FACE OF RETAINING WALL

WALL W65-10

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CHECKED BY ________ DATE ________
APPROVED BY ________ DATE ________

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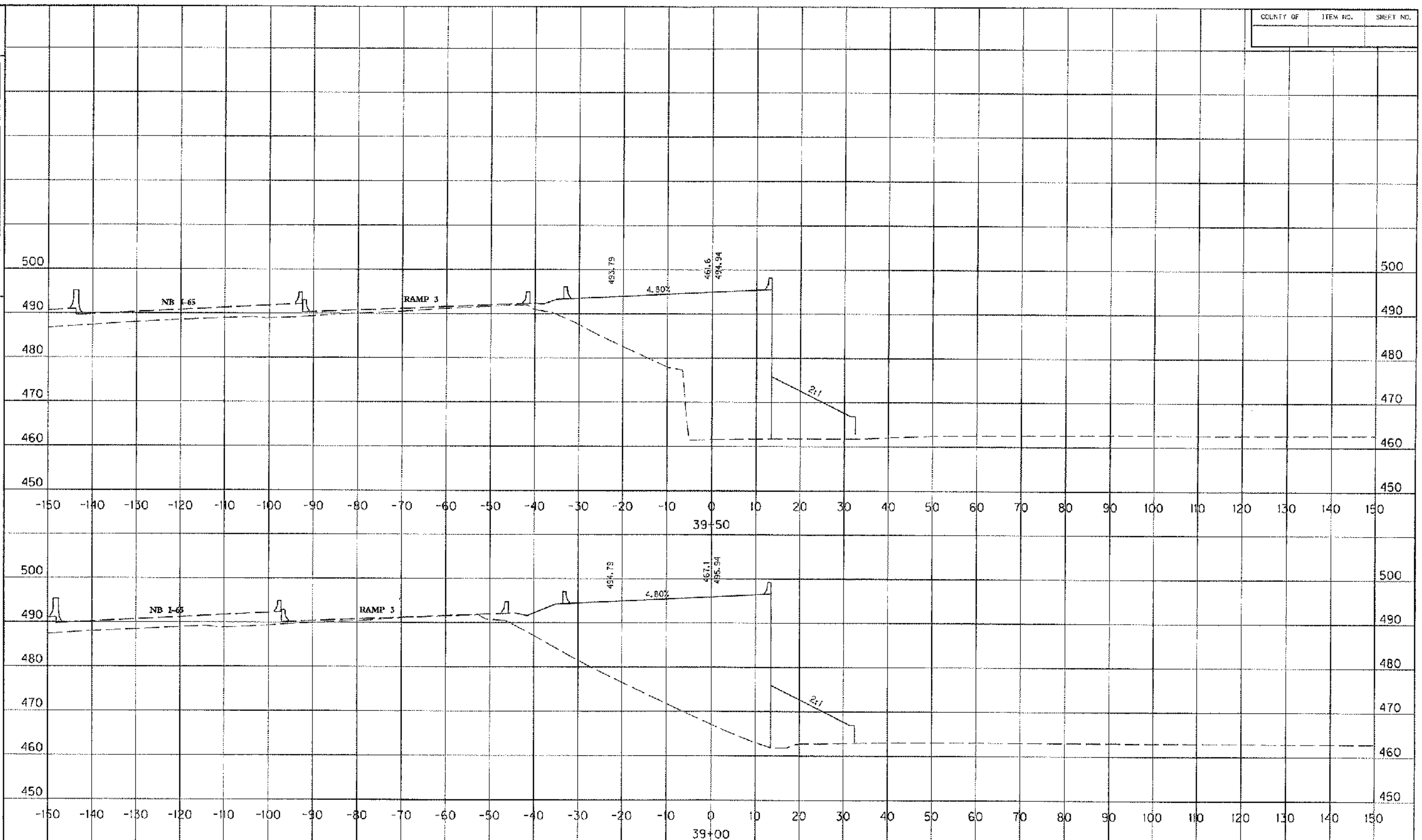

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THE OHIO RIVER BRIDGES

Kentucky Transportation Associates

SCALE: 1"=20'

RAMP 2
STA. 39+00 TO STA. 39+50

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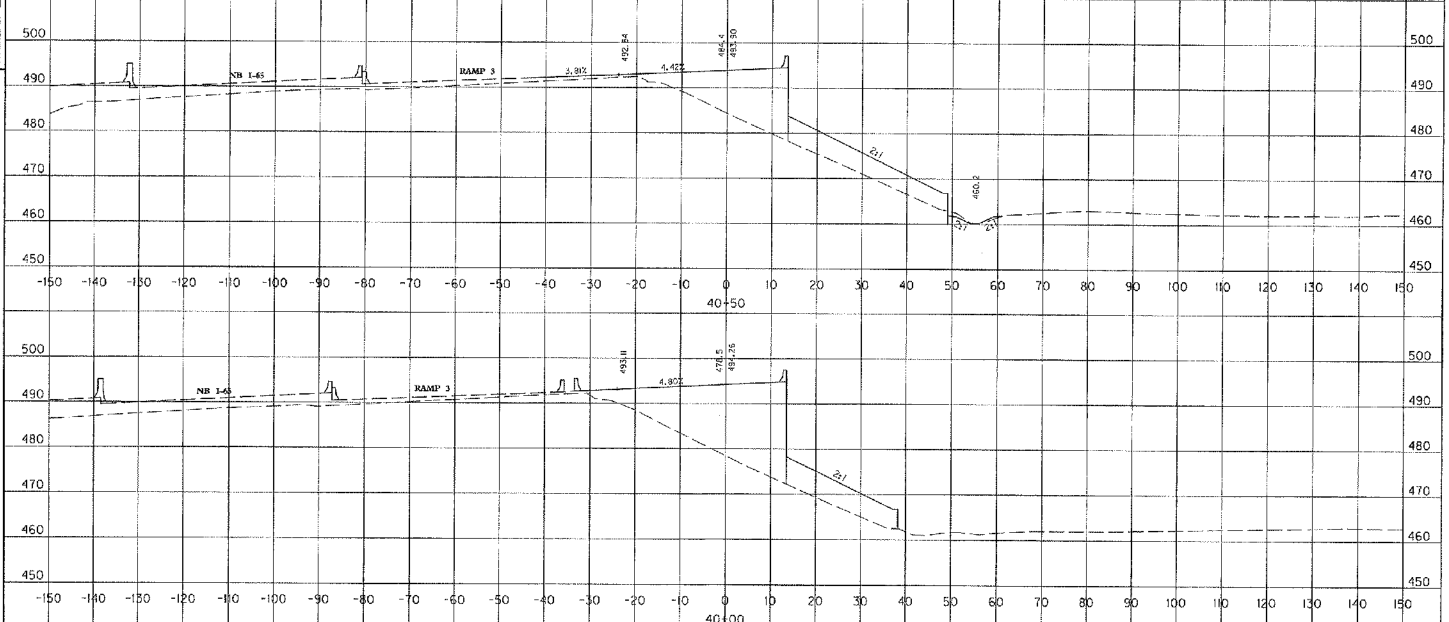

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3/5/07

THE OHIO RIVER BRIDGES

Kentucky Transportation Associates

SCALE: 1"=20'

RAMP 2
STA. 40+00 TO STA. 40+50

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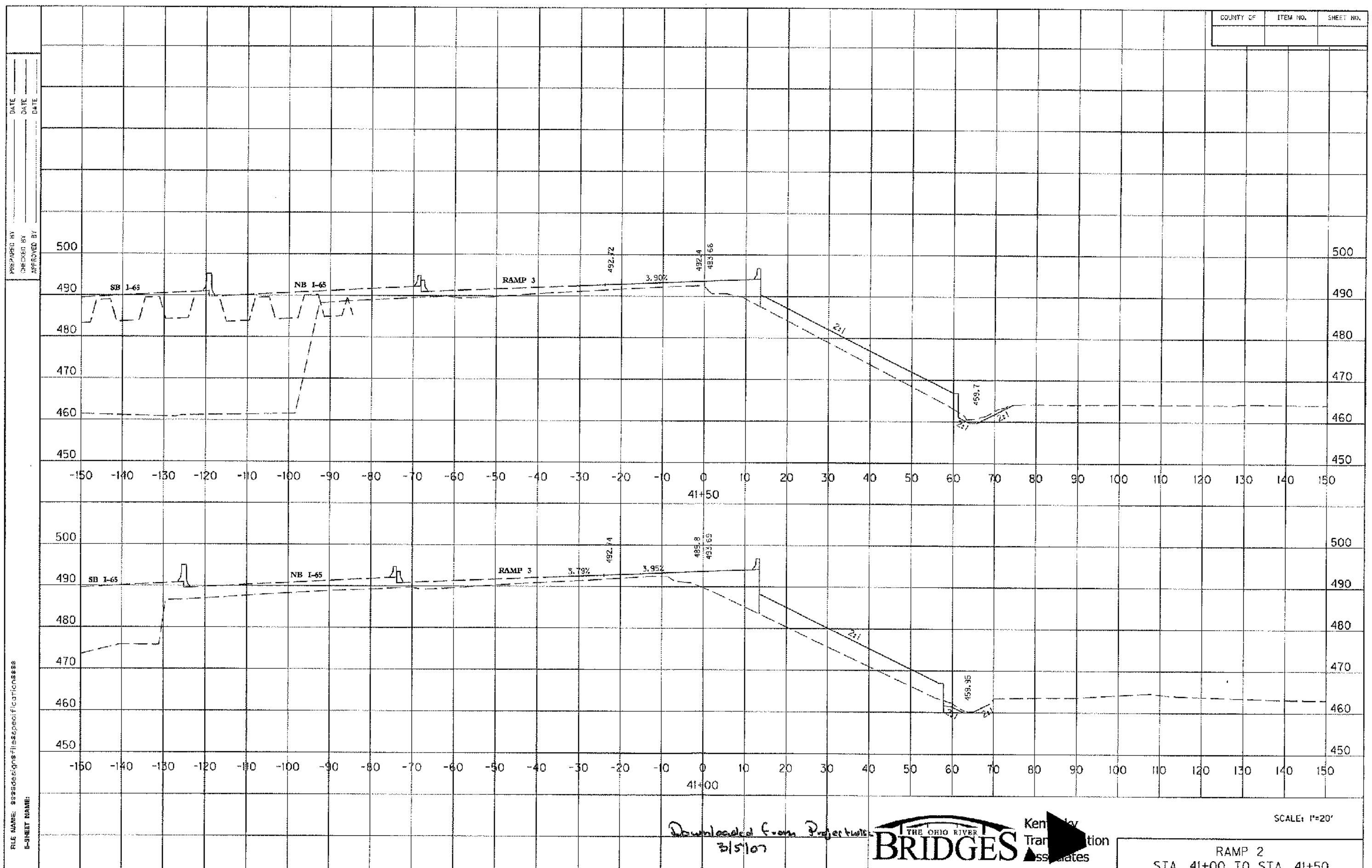

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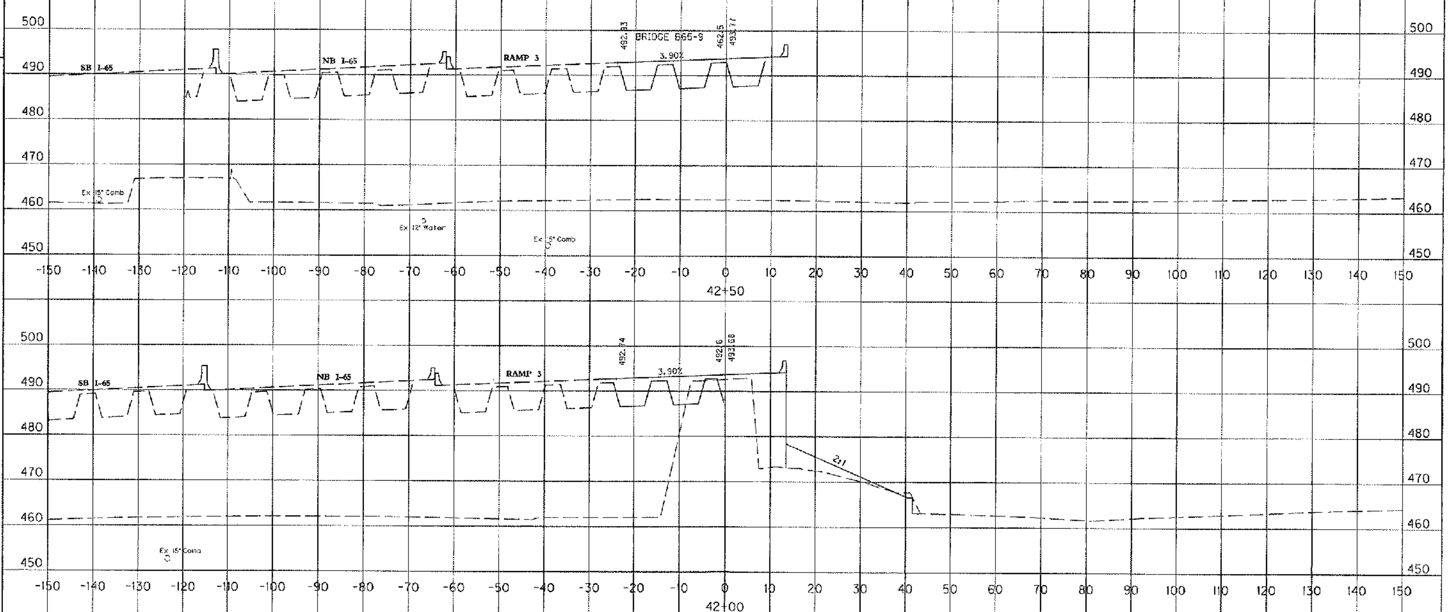

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THE OHIO RIVER BRIDGES

Kentucky Transportation Associates

SCALE: 1"=20'

RAMP 2
STA. 42+00 TO STA. 42+50

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Appendix C

Subsurface Data Sheets for MSE Wall Option

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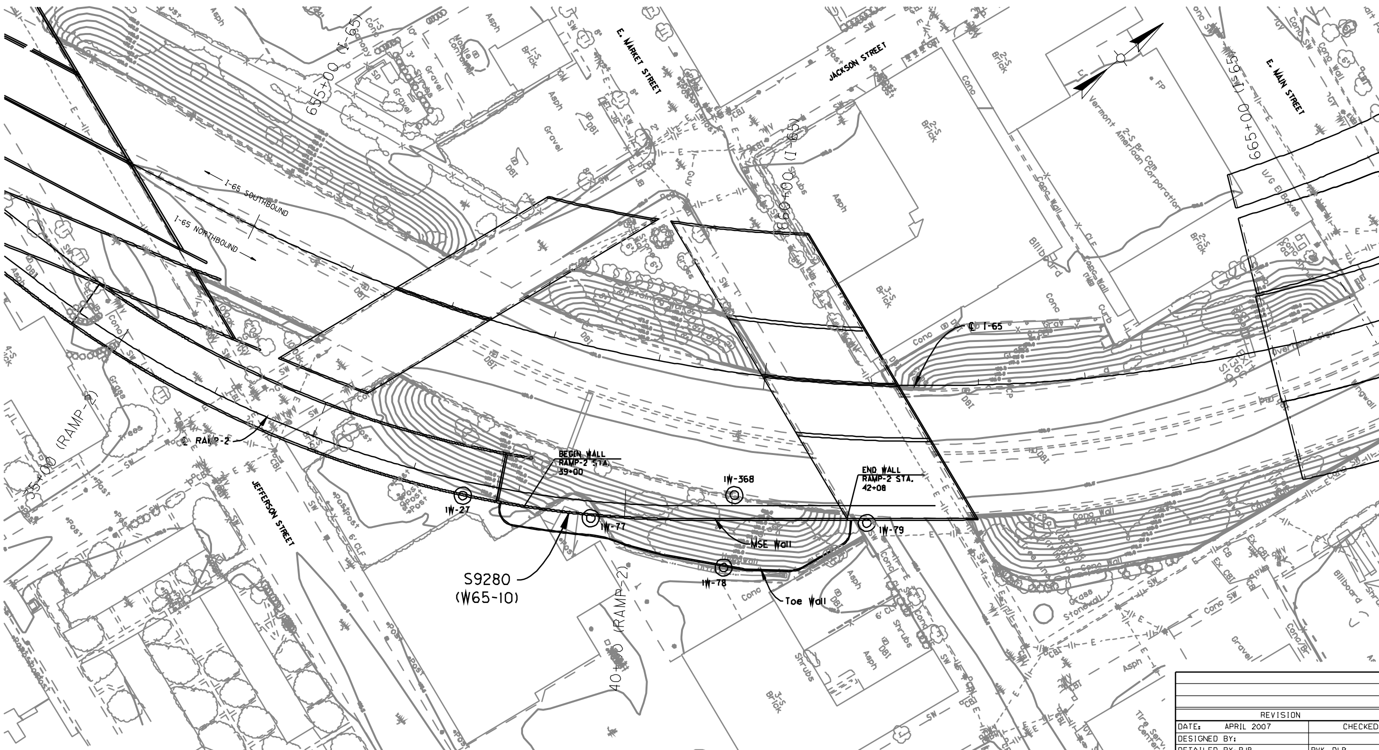

BORING LAYOUT

SCALE: 1"=40'

NOTE:

The information for the retaining wall layout used on this drawing was obtained through the Bentley ProjectWise Program from the Kentucky Transportation Associates Design Team on March 1, 2007.

LX2004130/PHASE1/S9280/MSE/S9280_LAYO.DGN

SHEET 1 OF 9

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) MSE RETAINING WALL

BORING LAYOUT

ITEM NUMBER
5-118.18/19

PREPARED BY

BRIDGES

Kentucky Transportation Associates

SHEET NO.

DRAWING NO.

GEOTECHNICAL NOTES

for MSE Walls

1. Design of the subject retaining structure should be in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

2. Based on the subsurface conditions encountered in borings performed along the wall alignment, the wall will be a soil bearing structure. It is recommended that the minimum wall embedment depth be in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

3. Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on clayey soils from Station 39+00 to 40+00 and shale (shot rock) embankment from Station 40+00 to 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clays and the friction angle of the shale. Based on a value of cohesion of 790 psf and a friction angle of 35 degrees for the alluvial clays and shale (shot rock) embankment materials, respectively, derived from unconfined compression test results and correlations of corrected SP N-values, the estimated nominal bearing capacity (qn) of the soils between Station 39+00 and 40+00 is on the order of 4,226 psf. The estimated nominal bearing capacity (qn) of the shale embankment between Station 40+00 and 42+08 ranges from 27,050 psf to 44,308 psf.

4. Construction of the planned retaining wall will require excavations at the toe of the existing interstate and ramp embankments as well as temporary excavations within the embankments themselves. The Contractor should evaluate the stability of the existing embankments and adjacent structures in conjunction with temporary excavations to verify that the planned excavation/bracing/shoring system maintains the stability of the highway embankment.

5. Temporary wall slopes and foundation excavations in soil shall be properly designed, or should be properly braced/shored to reduce the possibility of collapse and provide adequate safety to people working in or around the excavations. Bracing/shoring shall be performed in accordance with applicable federal, state and local guidelines.

6. At the writing of this report, a borrow source for embankment material has not been identified. The engineering analyses performed for the retaining wall options are based on estimated soil strength parameters for the retained fill and embankment materials. It is recommended that borrow material to be used for embankment construction meet the following minimum strength parameters.

Embankment Material		Retained Fill	
Total Stress	**Effective Stress**	**Total Stress**	**Effective Stress**
c = 1400 psf	$\bar{c}$ = 200 psf	c = 1400 psf	$\bar{c}$ = 170 psf
ϕ = 0°	$\bar{\phi}$ = 23°	ϕ = 0°	$\bar{\phi}$ = 27°
γ = 120 pcf	γ = 120 pcf	γ = 120 pcf	γ = 120 pcf

The retained fill material shall be placed in the entire area between the wall and a 1:1 (H:V) line sloping upward and away from the base of the heel of the wall for a CIP wall or the base of the reinforced material for a MSE wall to the top of the wall. Non-durable shales and fat clays (USCS classification of CH) should specifically be excluded from use within this zone.

The Contractor shall perform laboratory testing to confirm that the minimum total stress and effective stress strength parameters are equal to or greater than the above values per material type for each borrow area. The test results shall be submitted to the Engineer for approval.

7. Fill materials associated with interstate construction and/or previous development in the City of Louisville were encountered within each of the five borings drilled along the wall alignment. Because the structure site is located within an area of previous site grading and construction, the Contractor should anticipate encountering fill materials along the wall alignment. The excavations shall be deepened as necessary to provide an adequate bearing medium.

8. Granular embankment used as backfill and/or for over excavation and replacement shall be non-erodible and shall conform to the requirements of Section 805 of the current Kentucky Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The granular embankment material shall be wrapped with Type IV geotextile fabric meeting the requirements of Section 843 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction to provide separation from the clay embankment and/or foundation materials.

9. Soils exposed within the bottoms of footing trenches shall be observed for suitability by a geotechnical engineer or an engineering technician working under his/her direct supervision. Old fill or unsuitable material which might be encountered shall be removed. Areas disturbed by the excavation process should be restored utilizing proper compaction methods.

10. If soft, wet soils are encountered in the foundation excavations, they should be undercut a minimum depth of two (2) feet and backfilled to the design bearing elevation using non-erodible Granular Embankment conforming to Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The bottoms of the foundation excavations should be proof-rolled to restore the in-place density of any soil disturbed in the excavation process. Isolated zones of loose or wet soil may also be stabilized using KY size No. 2, 3, or 23 stone, as specified in Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

11. Based on the nominal bearing capacities (qn) outlined in Geotechnical Note 3 and a resistance factor (øb) of 0.67, the factored bearing capacity for the MSE wall option should be designed using the following factored bearing capacities (qR):

Station Interval	Factored Bearing Capacity (qR)*
Ramp 2 39+00 to 40+00	2,830 psf
Ramp 2 40+00 to 40+50	5,730 psf
Ramp 2 40+50 to 41+00	8,870 psf
Ramp 2 41+00 to 41+50	16,220 psf
Ramp 2 41+50 to 42+08	4,180 psf

*Applicable for no over excavation.

12. Based on eccentricity requirements outlined in Section 11.6.3.3 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition, it is recommended that the minimum reinforcing strap length used for design and construction of the MSE wall option conform to the guidelines outlined in the table below. However, the minimum strap length should not be less than eight (8) feet.

Guidelines For Minimum Reinforcement Strap Length for MSE Walls

Wall Height	Minimum Strap Length
29 ft $\leq$ H	0.7H
16 ft $\leq$ H < 29 ft	0.8H
12 ft $\leq$ H < 16 ft	0.9H
9 ft $\leq$ H < 12 ft	1.0H

This strap length may need to be increased by the wall designer depending on the results of the internal wall stability analyses. The wall designer should verify MSE wall stability against sliding, eccentricity and bearing capacity failure, based on the final wall design dimensions. The stability analyses should be performed in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

13. The minimum wall embedment shall be as specified in Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition, or two (2) feet, whichever is greater, as measured from the ground surface in front of the wall down to the base of the wall.

14. The internal design of the MSE wall shall be performed in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition. The pullout resistance shall be based on a ϕ = 34°, unless specific laboratory tests are conducted to obtain pullout design parameters.

15. It is recommended that the gradation of the reinforced soil conform to guidelines presented in the FHWA publication "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" and Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. During MSE wall construction this material should be wrapped with an engineering fabric (Geotextile) for separation. The engineering fabric should conform to the requirements of Section 843 of the current Kentucky Standard Specifications for Road and Bridge Construction.

16. Based on the loading conditions calculated for the MSE wall, and the factored bearing capacity of the existing foundation soils, it is recommended that portions of the wall foundation area be over-excavated and backfilled with granular embankment in an effort to spread the wall load over a large area and meet bearing capacity requirements. The areas requiring foundation soil modification and the type of modification recommended are presented in the table below.

Recommended Foundation Soil Modification for MSE Wall

Station Interval*	Maximum Wall Height (ft)	Bearing Capacity at the base of the Over Excavation**	Recommended Foundation Soil Modification
39+00 to 40+00	36.0 ft	5,670 psf (Alluvial Clay)	Over-excavate to a minimum depth of 8 feet below the base of the MSE wall and 8 feet horizontally beyond the perimeter (assuming a 1H:1V backfill slope). The excavation should be backfilled with geogrid reinforced granular embankment. (Geotechnical Note 8.)
40+00 to 40+50	24.0 ft	5,730 psf (Shale Embankment)	No soil modification required
40+50 to 41+00	17.5 ft	8,870 psf (Shale Embankment)	No soil modification required
41+00 to 41+50	12.0 ft	16,220 psf (Shale Embankment)	No soil modification required
41+50 to 42+08	26.5 ft	4,160 psf (Alluvial Clay)	Over-excavate to a minimum depth of 5 feet below the base of the MSE wall and 5 feet horizontally beyond the perimeter (assuming a 1H:1V backfill slope). The excavation should be backfilled with geogrid reinforced granular embankment. (Geotechnical Note 8.)

*Ramp 2 Stationing
**The values presented in the table include an increase in bearing capacity for over excavation and an increase or decrease for a two-layer system as applicable.

17. Additional horizontal over-excavation of the foundation soils is recommended in select areas as outlined in Geotechnical Note 16 to spread the applied wall loads over a larger area. Biaxial geogrid reinforcement shall be incorporated into the granular replacement of the over excavated foundation soils within these areas to assist load transfer. The geogrid reinforcement shall be placed between one foot layers of compacted non-erodible granular embankment. The biaxial geogrid shall extend the full width and length of the area requiring horizontal over excavation. At a minimum, the biaxial geogrid reinforcement shall exhibit the following Machine Direction (MD) values:

Index Property	MD Value*
2% Junction Tension Modulus in Use	18,200 lb/ft
Junction Strength in Use at 2% Strain	370 lb/ft

* In accordance with GRI-GG2-87

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-65 | CROSSING S9280 (W65-10) MSE RETAINING WALL

GEOTECHNICAL NOTES

PREPARED BY

ITEM NUMBER
5-118.18/19

SHEET NO.
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GEOTECHNICAL NOTES

for MSE Walls

18. Based on settlement calculations performed along the wall alignment, settlements up to 8.7 inches can be expected. Differential settlements on the order of 4.7 inches occurring over a wall length of 5 feet (1/13) should be anticipated and considered in the design of the MSE wall. The wall designer should select the panels and size the joint widths between the wall panels to accommodate the anticipated settlements. It is FMSM's understanding that the permanent fascia will not be installed until an appropriate amount of settlement has occurred as indicated in Geotechnical Note 21. If this cannot be done, ground improvement techniques such as, additional over excavation and replacement, or use of stone columns may be warranted to reduce the anticipated settlement. The over-excavation limits recommended in Geotechnical Note 16 are required to modify foundation conditions to achieve adequate bearing capacity. Additional over-excavation and replacement of soils with select embankment would be required to reduce the magnitude of settlement.

19. It is recommended that vertical slip joints be incorporated into the design and construction of the MSE wall at the following locations in conjunction with a "step" in the wall bearing elevation. It is recommended that vertical slip joints also be constructed at any additional 'steps' introduced into the planned bearing elevation by the wall Designer.

Ramp 2 Station

40+00
40+50
41+00
41+50

20. If the placement of an obstruction in the wall reinforcement zone such as drainage structures, signal or sign foundations, guardrail posts, or the bridge foundation system (piles or drilled shafts/caissons) cannot be avoided, the design of the wall near the obstruction shall be modified using one of the following alternatives:

   a. Place a structural frame (collar or yoke) around the obstruction which is capable of carrying the load from the reinforcement in front of the obstruction to the reinforcement connected to the structural frame behind the obstruction.

   b. If the soil reinforcements consist of discrete strips or bar mats rather than continuous sheets, depending on the size of the obstruction, it may be possible to splay the reinforcements around the obstruction.

   c. Reinforcement layers shall not be structurally connected to any foundation elements.

21. Settlement analyses were performed along the planned wall alignment in order to develop settlement profiles for the proposed structure. It is FMSM's understanding that the wall will be constructed and allowed to settle prior to the attachment of the permanent fascia. Time rate of settlement calculations indicate 90 percent of primary consolidation will occur in about 189 days (27 weeks). In order to monitor the settlement of the wall, settlement platforms shall be furnished and installed by the Contractor at the following approximate locations. Installation of the platforms shall be in accordance with Section 216 of the current Standard Specifications for Road and Bridge Construction and Standard Drawing RGX-015.

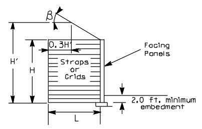
Ramp 2 Station

39+50, 10' Rt.
40+25, 10' Rt.
40+75, 10' Rt.
41+25, 10' Rt.
41+75, 10' Rt.

The exact locations shall be determined by the Engineer and a representative of the Division of Structural Design, Geotechnical Branch once wall layouts have been finalized. Settlement readings shall begin with an initial reading upon placement of the platform prior to beginning embankment/retaining wall construction. Readings shall continue periodically during and following completion of the subject roadway embankments and retaining structures. The settlement platforms shall be left in place for future readings after the project is completed. Necessary forms for recording settlement measurements can be obtained from the Geotechnical Branch at the request of the Engineer. The Engineer will be responsible for reading and recording the settlement data. The Geotechnical Branch will be responsible for evaluation of the actual settlement data. Installation of the permanent fascia shall not begin until the Geotechnical Branch has determined that an adequate percent of consolidation of the foundation soils has been achieved. The Contractor shall be responsible for replacing all damaged settlement platforms at no extra cost.

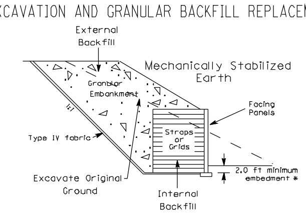
Use the following soil strength parameters for design:

	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
Internal Backfill			
(in reinforced volume)	0	34	115
Retained Fill			
Soil Embankment	170	27	120
Granular Embankment	0	38	120
Foundation Soils			
Existing Clay Soils	150	32	128
Existing Embankment	0	35	122
Granular Replacement	0	38	120

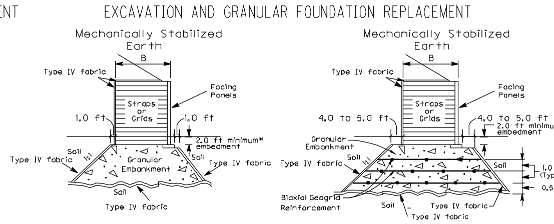


The minimum reinforcment length (L) shall be the greater of:
L > 0.7 H' (Where H' is the effective wall height)
L > 8 ft
H' = H/(1-0.3 tan β) for sloping backfill
H'=H for level backfill

EXCAVATION AND GRANULAR BACKFILL REPLACEMENT



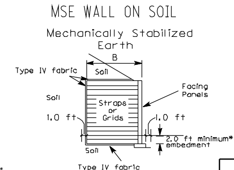
EXCAVATION AND GRANULAR FOUNDATION REPLACEMENT



Type IV Fabric required only where there is a soil-granular material interface.

Type IV Fabric required only where there is a soil-granular material interface.

MSE WALL ON SOIL



* - Unless Otherwise Noted

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-65 | CROSSING S9280 (W65-10) MSE RETAINING WALL

GEOTECHNICAL NOTES

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/19

LX2004130/PHASE1/S9280/MSE/S9280_GEONTS2.DGN

GEOTECHNICAL NOTES

for Toe Wall

1. Design of the subject retaining structure should be in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

2. Based on the subsurface conditions encountered in borings performed along the wall alignment, the wall will be a soil bearing structure. It is recommended that the minimum wall embedment depth be in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

3. Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on clayey soils from Station 39+00 to 40+00 and shale (shot rock) embankment from Station 40+00 to 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clays and the friction angle of the shale. Based on a value of cohesion of 790 psf and a friction angle of 35 degrees for the alluvial clays and shale (shot rock) embankment materials, respectively, derived from unconfined compression test results and correlations of corrected SP N-values, the estimated nominal bearing capacity (qn) of the soils between Station 39+00 and 40+00 is on the order of 4,226 psf. The estimated nominal bearing capacity (qn) of the shale embankment between Station 40+00 and 42+08 ranges from 27,050 psf to 44,308 psf.

4. Construction of the planned retaining wall will require excavations at the toe of the existing interstate and ramp embankments as well as temporary excavations within the embankments themselves. The Contractor should evaluate the stability of the existing embankments and adjacent structures in conjunction with temporary excavations to verify that the planned excavation/bracing/shoring system maintains the stability of the highway embankment.

5. Temporary wall slopes and foundation excavations in soil shall be properly designed, or should be properly braced/shored to reduce the possibility of collapse and provide adequate safety to people working in or around the excavations. Bracing/shoring shall be performed in accordance with applicable federal, state and local guidelines.

6. At the writing of this report, a borrow source for embankment material has not been identified. The engineering analyses performed for the retaining wall options are based on estimated soil strength parameters for the retained fill and embankment materials. It is recommended that borrow material to be used for embankment construction meet the following minimum strength parameters.

Embankment Material		Retained Fill	
Total Stress	**Effective Stress**	**Total Stress**	**Effective Stress**
c = 1400 psf	$\bar{c}$ = 200 psf	c = 1400 psf	$\bar{c}$ = 170 psf
ϕ = 0°	$\bar{\phi}$ = 23°	ϕ = 0°	$\bar{\phi}$ = 27°
γ = 120 pcf	γ = 120 pcf	γ = 120 pcf	γ = 120 pcf

The retained fill material shall be placed in the entire area between the wall and a 1:1 (H:V) line sloping upward and away from the base of the heel of the wall for a CIP wall or the base of the reinforced material for a MSE wall to the top of the wall. Non-durable shales and fat clays (USCS classification of CH) should specifically be excluded from use within this zone.

The Contractor shall perform laboratory testing to confirm that the minimum total stress and effective stress strength parameters are equal to or greater than the above values per material type for each borrow area. The test results shall be submitted to the Engineer for approval.

7. Fill materials associated with interstate construction and/or previous development in the City of Louisville were encountered within each of the five borings drilled along the wall alignment. Because the structure site is located within an area of previous site grading and construction, the Contractor should anticipate encountering fill materials along the wall alignment. The excavations shall be deepened as necessary to provide an adequate bearing medium.

8. Granular embankment used as backfill and/or for over excavation and replacement shall be non-erodible and shall conform to the requirements of Section 805 of the current Kentucky Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The granular embankment material shall be wrapped with Type IV geotextile fabric meeting the requirements of Section 843 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction to provide separation from the clay embankment and/or foundation materials.

9. Soils exposed within the bottoms of footing trenches shall be observed for suitability by a geotechnical engineer or an engineering technician working under his/her direct supervision. Old fill or unsuitable material which might be encountered shall be removed. Areas disturbed by the excavation process should be restored utilizing proper compaction methods.

10. If soft, wet soils are encountered in the foundation excavations, they should be undercut a minimum depth of two (2) feet and backfilled to the design bearing elevation using non-erodible Granular Embankment conforming to Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The bottoms of the foundation excavations should be proof-rolled to restore the in-place density of any soil disturbed in the excavation process. Isolated zones of loose or wet soil may also be stabilized using KY size No. 2, 3, or 23 stone, as specified in Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

11. Based on the depth to bedrock and the anticipated wall loading, it is recommended that the retaining wall be supported by a yielding foundation system. The allowable bearing capacity of the underlying soil material is 2,230 pounds per square foot.

12. To maintain an acceptable factor for the CDRsliding, it is recommended that a minimum of one foot of material be excavated below the wall footprint and backfilled with non-erodible granular embankment as defined in Geotechnical Note 8.

13. The minimum wall embedment shall be two feet, as measured from the ground surface in front of the wall down to the base of the footing.

14. Retaining wall stability analyses indicate the geometry for a six foot tall standard gravity wall (KYTC Standard Drawing RGX-002-07) will meet the minimum factor for CDRsliding based on the AASHTO LRFD Bridge Design Specifications, Fourth Edition. If a gravity type retaining wall is chosen, the foundation soils will need to be over excavated and replaced with granular embankment (see Geotechnical Note 12), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Geotechnical Note 8. As another option, a gravity type retaining wall can be used with random backfill provided that the foundation soils are over excavated and replaced with granular embankment and the base width is widened to 7.2 feet.

Using a phi angle of 38 degrees, a wall height measuring 6 feet, a base width measuring 3 feet, an angle between back of the wall and vertical equal to 14.0 degrees, a friction angle between the back of the wall and the granular backfill equal to 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	41 psf
3:1(H:V)	53 psf
2:1(H:V)	64 psf

The backfill shall be placed in the entire area between the wall and a 1:1(H:V) line sloping upward away from the base of the heel of the wall to the top of the wall. Type IV geotextile fabric shall be placed on the 1:1(H:V) slope to reduce the infiltration of fines into the granular material behind the wall and help prevent clogging of the drainage system. The drainage system shall consist of 4 inch diameter pipe with weepholes installed at locations as indicated by KYTC Standard Drawing RGX 002-07 or by the Designer; and/or perforated pipe installed at the base of the wall and 'daylighted' to promote 'dewatering' of the granular backfill.

15. Retaining wall stability analyses indicate a cantilever-type retaining wall measuring six feet in height can meet the minimum factor for CDRsliding and eccentricity provided that the foundation soils are over excavated and replaced with granular embankment (see Geotechnical Note 12). In addition, the footing width will need to be increased to 4.8 feet (0.8H), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Geotechnical Note 8.

Using a phi angle of 38 degrees, a vertical back of wall, friction angle between the back of the wall and backfill of 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

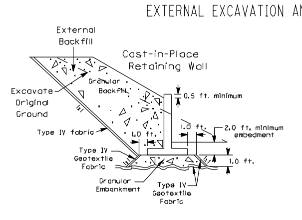
Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	26 psf
3:1(H:V)	33 psf
2:1(H:V)	39 psf

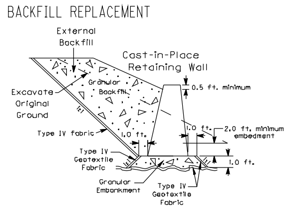
Drainage systems consisting of free draining material and filter fabric shall be placed directly behind the wall and be a minimum thickness of two (2) feet. Use of filter fabric will help reduce the infiltration of fines into the granular material behind the wall and help reduce clogging of the drainage system. In addition, weep holes shall also be provided in the design of the walls. If a drainage system is not provided, the design shall incorporate full hydrostatic forces behind the wall.

Use the following soil strength parameters for design:

	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
External Backfill			
Soil Embankment	200	23	120
Granular Embankment	0	38	120
Foundation Soils			
Existing	150	32	128
Granular Replacement	0	38	120

EXTERNAL EXCAVATION AND BACKFILL REPLACEMENT





REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE: I-65
CROSSING: S9280 (W65-10) TOE WALL

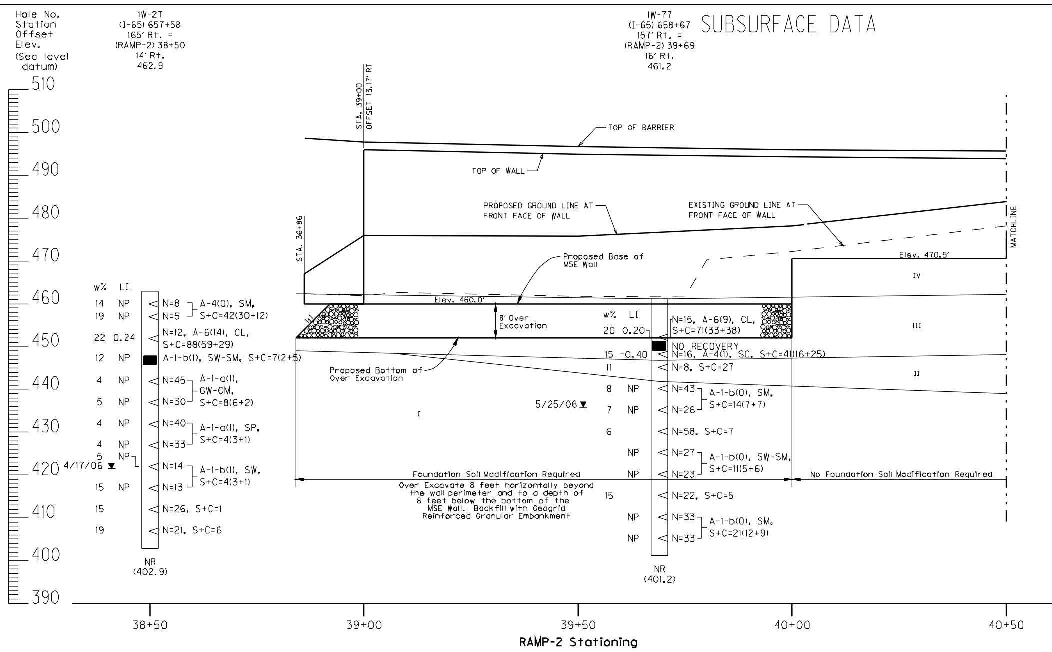
GEOTECHNICAL NOTES

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/19

SUBSURFACE DATA



For clarity, the planned toe wall is not shown in the elevation view depicted on this sheet.

SUMMARY OF ESTIMATED SETTLEMENT PARAMETERS	
SOIL LAYER	SETTLEMENT PARAMETERS
IV	C' = 65 γ = 122 pcf
III	e_0 = 0.765 C_c = 0.198 C_r = 0.033 P_c = 1,000 psf γ = 128 pcf
II	C' = 39 γ = 113 pcf
I	C' = 55 γ = 124 pcf

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SHEET LOCATION:
FILE NAME: $$$$designfilesspecification$$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) MSE RETAINING WALL

BORING LOGS AND PROFILE

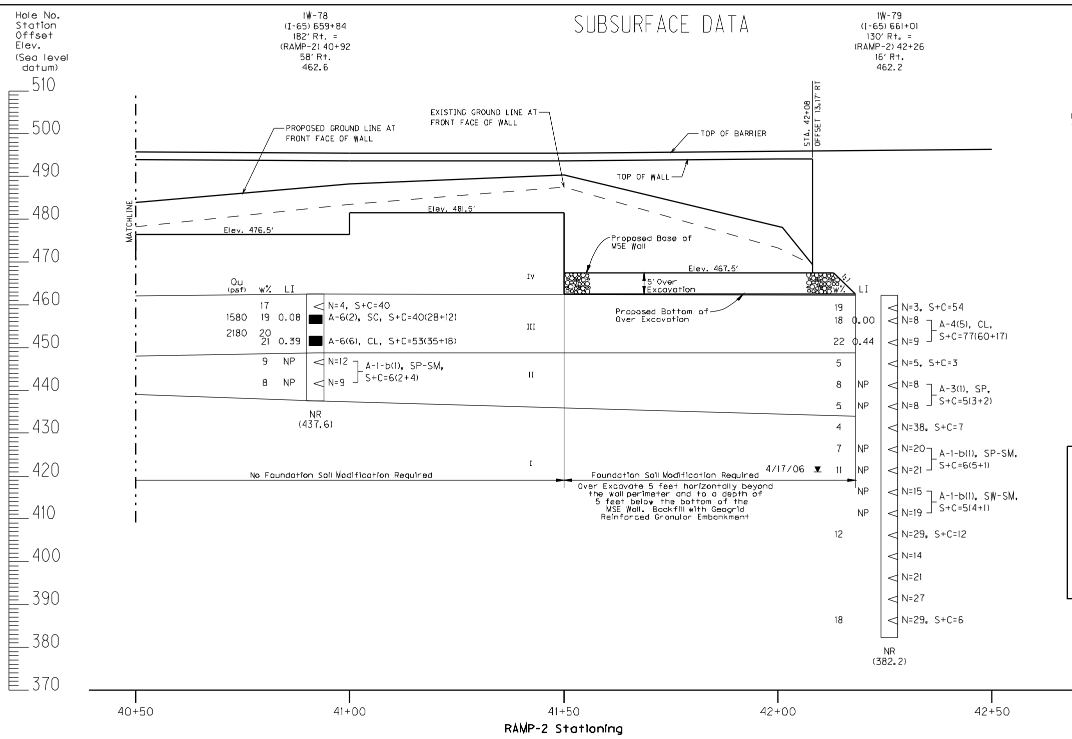
PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/.19

LX2004130/PHASE1/S9280/MSE/S9280_PROF1.DGN

SUBSURFACE DATA



For clarity, the planned toe wall is not shown in the elevation view depicted on this sheet.

SUMMARY OF ESTIMATED SETTLEMENT PARAMETERS	
SOIL LAYER	SETTLEMENT PARAMETERS
IV	C' = 65 γ = 122 pcf
III	e_0 = 0.765 C_c = 0.198 C_r = 0.033 P_c = 1,000 psf γ = 128 pcf
II	C' = 39 γ = 113 pcf
I	C' = 55 γ = 124 pcf

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SCALE: 1" = 10'

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY **JEFFERSON**

ROUTE **I-65** CROSSING **S9280 (W65-10) MSE RETAINING WALL**

BORING LOGS AND PROFILE

PREPARED BY BRIDGES · Kentucky Transportation Associates

SHEET NO. DRAWING NO.

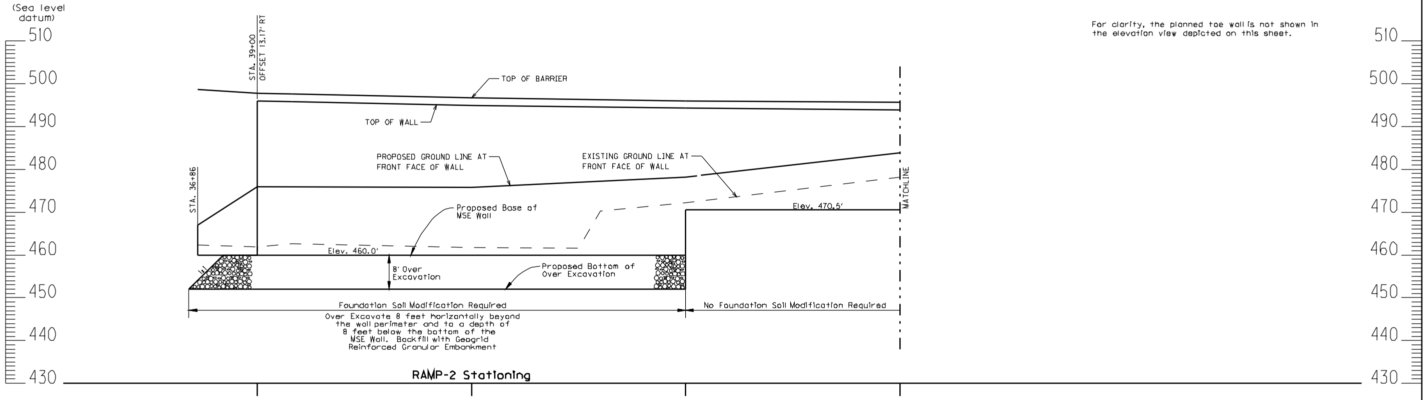
ITEM NUMBER 5-118.18/19

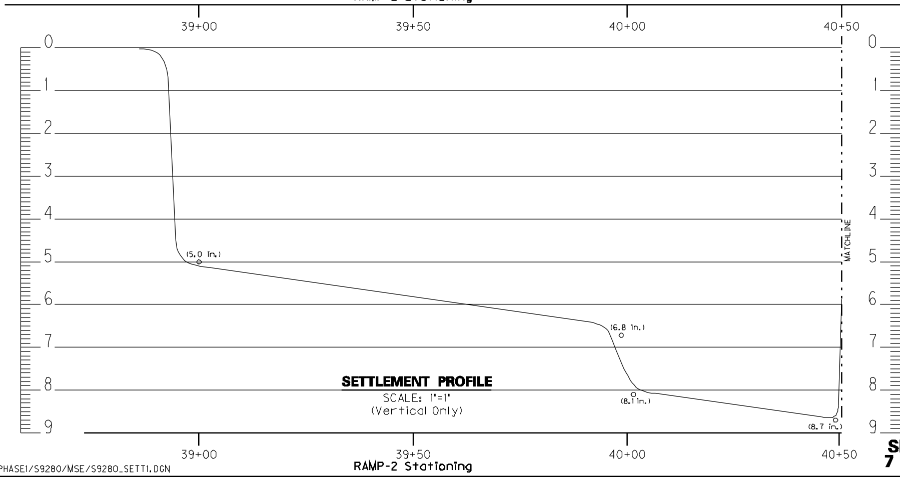
SHEET LOCATION: FILE NAME: $$$$designfilesspecification$$$$ USERNAME: $$$$USER$$$$ DATE: $$$$DATE$$$$ E-SHEET NAME:

LX2004130/PHASE1/S9280/MSE/S9280_PROF2.DGN

SUBSURFACE DATA

For clarity, the planned toe wall is not shown in the elevation view depicted on this sheet.





SETTLEMENT PROFILE

SCALE: 1"=1"
(Vertical Only)

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley ProjectWise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) MSE RETAINING WALL

SETTLEMENT PROFILE

PREPARED BY

The Ohio River BRIDGES — Kentucky Transportation Associates

SHEET NO.

DRAWING NO.

SHEET 7 OF 9

ITEM NUMBER
5-118.18/19

LX2004130/PHASE1/S9280/MSE/S9280_SETT1.DGN

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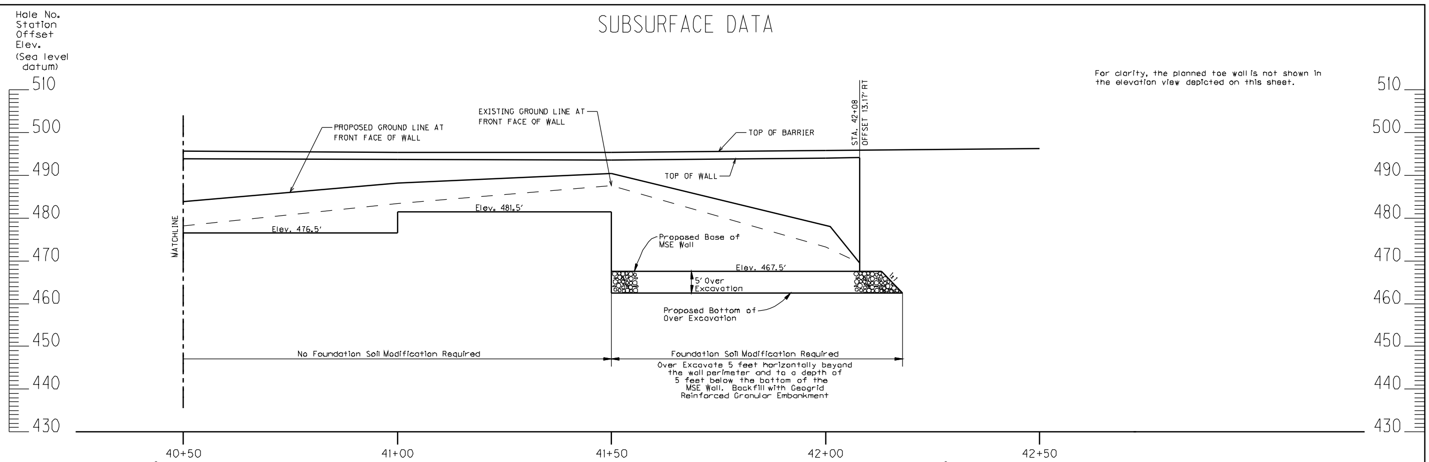

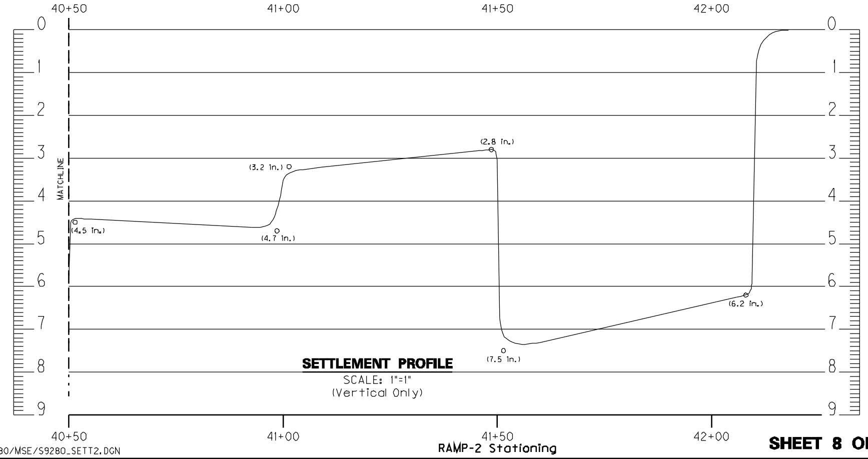

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley ProjectWise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) MSE RETAINING WALL

SETTLEMENT PROFILE

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/19

SHEET 8 OF 9

LX2004130/PHASE1/S9280/MSE/S9280_SETT2.DGN

SHEET LOCATION:
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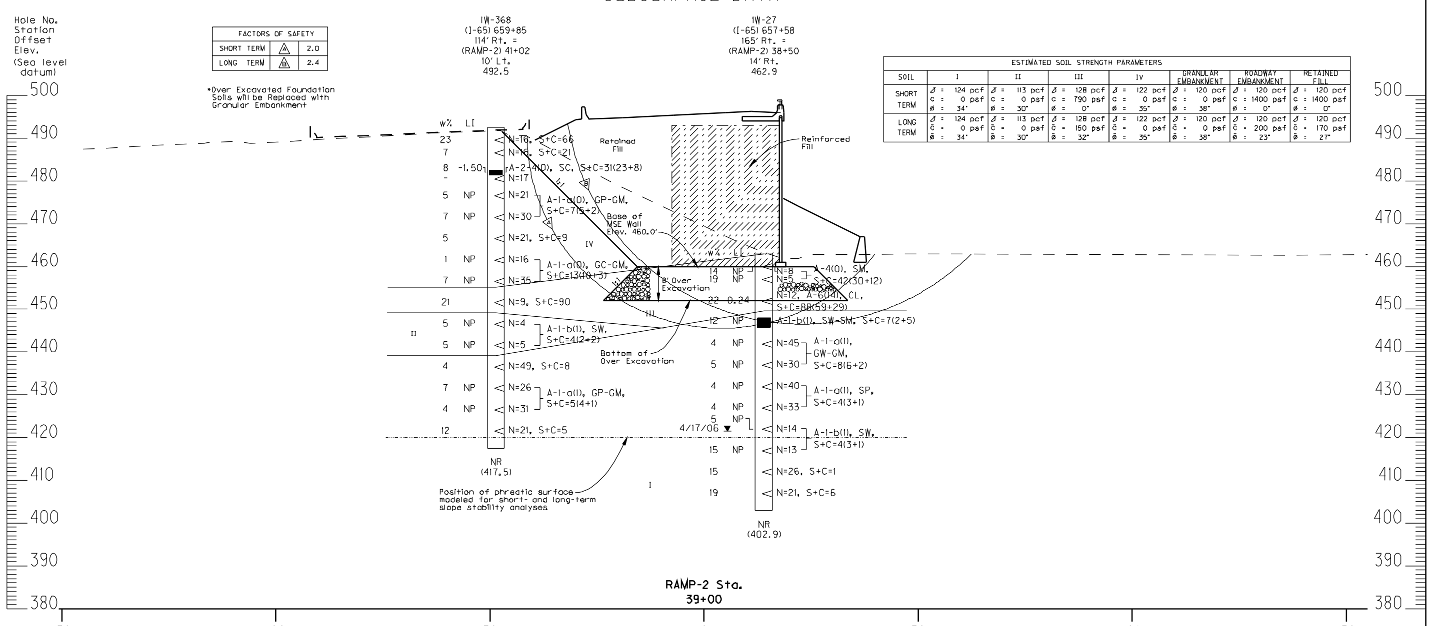
E-SHEET NAME:

SUBSURFACE DATA



FACTORS OF SAFETY		
SHORT TERM	A	2.0
LONG TERM	B	2.4

*Over Excavated Foundation Soils will be Replaced with Granular Embankment

ESTIMATED SOIL STRENGTH PARAMETERS							
SOIL	I	II	III	IV	GRANULAR EMBANKMENT	ROADWAY EMBANKMENT	RETAINED FILL
SHORT TERM	γ = 124 pcf c = 0 psf ø = 34°	γ = 113 pcf c = 0 psf ø = 30°	γ = 128 pcf c = 790 psf ø = 0°	γ = 122 pcf c = 0 psf ø = 35°	γ = 120 pcf c = 0 psf ø = 38°	γ = 120 pcf c = 1400 psf ø = 0°	γ = 120 pcf c = 1400 psf ø = 0°
LONG TERM	γ = 124 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 34°	γ = 113 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 30°	γ = 128 pcf $\bar{c}$ = 150 psf $\bar{ø}$ = 32°	γ = 122 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 35°	γ = 120 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 38°	γ = 120 pcf $\bar{c}$ = 200 psf $\bar{ø}$ = 23°	γ = 120 pcf $\bar{c}$ = 170 psf $\bar{ø}$ = 27°

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall cross-section used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PKV, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE: I-65
CROSSING: S9280 (W65-10) MSE RETAINING WALL

STABILITY SECTION

PREPARED BY
The Ohio River BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

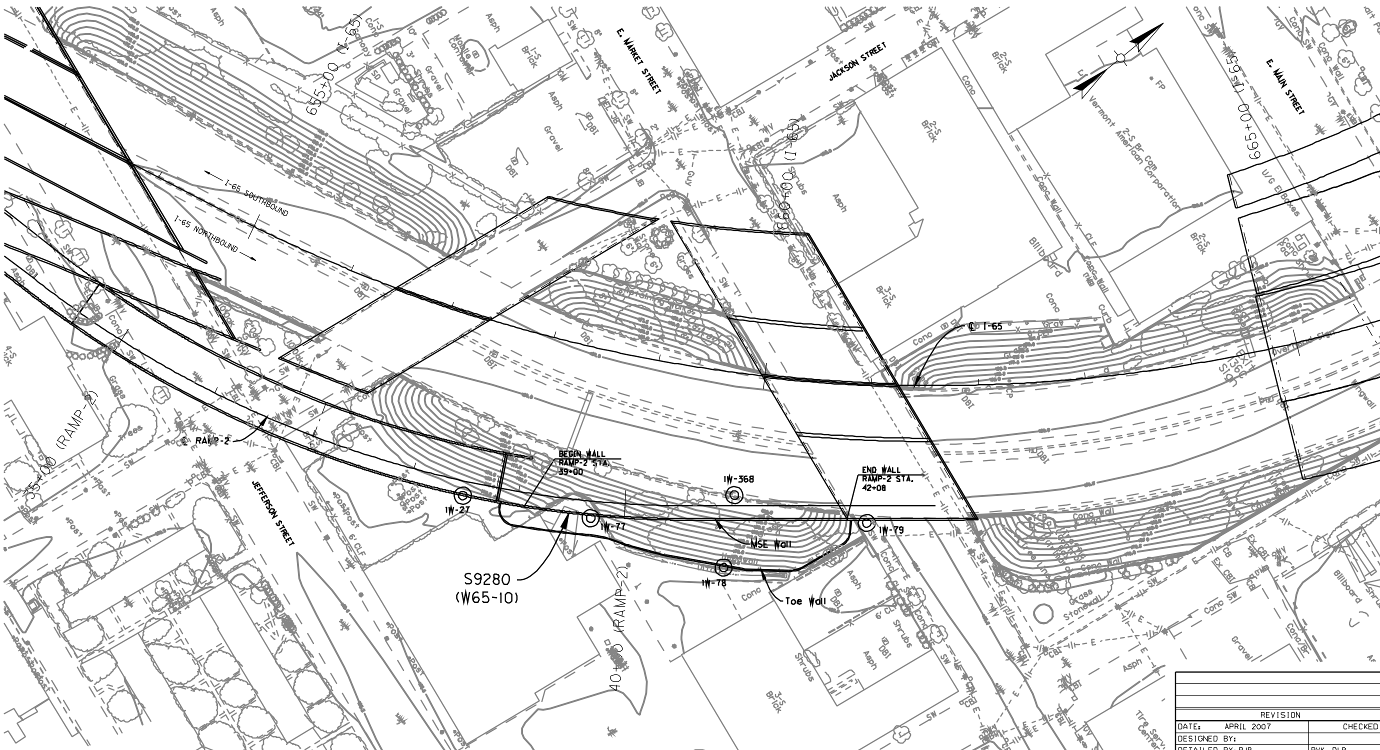
ITEM NUMBER
5-118.18/.19

LX2004130/PHASE1/S9280/MSE/S9280_STAB.DGN

SHEET LOCATION:
FILE NAME: $$$$designfile$specification$$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

Appendix D

Subsurface Data Sheets for CIP Cantilever Wall Option



BORING LAYOUT
SCALE: 1"=40'

NOTE:
The information for the retaining wall layout used on this drawing was obtained through the Bentley ProjectWise Program from the Kentucky Transportation Associates Design Team on March 1, 2007.

LX2004130/PHASE1/S9280/CIP/S9280_LAYO.DGN

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE **I-65** CROSSING **S9280 (W65-10) CIP RETAINING WALL**

BORING LAYOUT

PREPARED BY
BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/19

SHEET 1 OF 7

SHEET LOCATION:
FILE NAME: $$$$designfilespecification$$$$
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SHEET LOCATION:

FILE NAME: $$$design$file$specification$$$

USERNAME: $$$USER$$$$

DATE: $$$DATE$$$$

E-SHEET NAME:

GEOTECHNICAL NOTES

for CIP Cantilever Retaining Walls

1. Design of the subject retaining structure should be in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

2. Based on the subsurface conditions encountered in borings performed along the wall alignment, the wall will be a soil bearing structure. It is recommended that the minimum wall embedment depth be in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

3. Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on clayey soils from Station 39+00 to 40+00 and shale (shot rock) embankment from Station 40+00 to 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clays and the friction angle of the shale. Based on a value of cohesion of 790 psf and a friction angle of 35 degrees for the alluvial clays and shale (shot rock) embankment materials, respectively, derived from unconfined compression test results and correlations of corrected SP N-values, the estimated nominal bearing capacity (qn) of the soils between Station 39+00 and 40+00 is on the order of 4,226 psf. The estimated nominal bearing capacity (qn) of the shale embankment between Station 40+00 and 42+08 ranges from 27,050 psf to 44,308 psf.

4. Construction of the planned retaining wall will require excavations at the toe of the existing interstate and ramp embankments as well as temporary excavations within the embankments themselves. The Contractor should evaluate the stability of the existing embankments and adjacent structures in conjunction with temporary excavations to verify that the planned excavation/bracing/shoring system maintains the stability of the highway embankment.

5. Temporary wall slopes and foundation excavations in soil shall be properly designed, or should be properly braced/shored to reduce the possibility of collapse and provide adequate safety to people working in or around the excavations. Bracing/shoring shall be performed in accordance with applicable federal, state and local guidelines.

6. At the writing of this report, a borrow source for embankment material has not been identified. The engineering analyses performed for the retaining wall options are based on estimated soil strength parameters for the retained fill and embankment materials. It is recommended that borrow material to be used for embankment construction meet the following minimum strength parameters.

Embankment Material		Retained Fill	
Total Stress	**Effective Stress**	**Total Stress**	**Effective Stress**
c = 1400 psf	c̄ = 200 psf	c = 1400 psf	c̄ = 170 psf
ø = 0°	ø̄ = 23°	ø = 0°	ø̄ = 27°
γ = 120 pcf	γ = 120 pcf	γ = 120 pcf	γ = 120 pcf

The retained fill material shall be placed in the entire area between the wall and a 1:1 (H:V) line sloping upward and away from the base of the heel of the wall for a CIP wall or the base of the reinforced material for a MSE wall to the top of the wall. Non-durable shales and fat clays (USCS classification of CH) should specifically be excluded from use within this zone.

The Contractor shall perform laboratory testing to confirm that the minimum total stress and effective stress strength parameters are equal to or greater than the above values per material type for each borrow area. The test results shall be submitted to the Engineer for approval.

7. Fill materials associated with interstate construction and/or previous development in the City of Louisville were encountered within each of the five borings drilled along the wall alignment. Because the structure site is located within an area of previous site grading and construction, the Contractor should anticipate encountering fill materials along the wall alignment. The excavations shall be deepened as necessary to provide an adequate bearing medium.

8. Granular embankment used as backfill and/or for over excavation and replacement shall be non-erodible and shall conform to the requirements of Section 805 of the current Kentucky Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The granular embankment material shall be wrapped with Type IV geotextile fabric meeting the requirements of Section 843 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction to provide separation from the clay embankment and/or foundation materials.

9. Soils exposed within the bottoms of footing trenches shall be observed for suitability by a geotechnical engineer or an engineering technician working under his/her direct supervision. Old fill or unsuitable material which might be encountered shall be removed. Areas disturbed by the excavation process should be restored utilizing proper compaction methods.

10. If soft, wet soils are encountered in the foundation excavations, they should be undercut a minimum depth of two (2) feet and backfilled to the design bearing elevation using non-erodible Granular Embankment conforming to Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The bottoms of the foundation excavations should be proof-rolled to restore the in-place density of any soil disturbed in the excavation process. Isolated zones of loose or wet soil may also be stabilized using KY size No. 2, 3, or 23 stone, as specified in Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

11. The minimum wall embedment should be three (3) feet as measured from the ground surface in front of the wall to the base of the footing to provide approximately one (1) foot of soil cover over the wall footing.

12. Backfill behind the wall can consist of retained fill as defined in Geotechnical Note 6 or non-erodible granular embankment as defined in Geotechnical Note 8. Coefficients of active earth pressure (Ka) were determined based on Coulomb earth pressure theory using phi angles of 27 and 38 degrees, a vertical back of wall, and friction angles between the back of the wall and backfill of 17 and 29 degrees. Based on a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

	Common Backfill (ø = 27°)		Granular Embankment (ø = 38°)	
Slope of Backfill	**Coefficient of Active Earth Earth Pressure (K_a)**	**Equivalent Fluid Pressure Per Linear Foot**	**Coefficient of Active Earth Earth Pressure (K_a)**	**Equivalent Fluid Pressure Per Linear Foot**
Level	0.335	40 psf	0.218	26 psf
3:1(H:V)	0.464	56 psf	0.274	33 psf
2:1(H:V)	0.714	86 psf	0.323	39 psf

Drainage systems consisting of free draining material and filter fabric shall be placed directly behind the wall and be a minimum thickness of two feet. Use of filter fabric will help reduce the infiltration of fines into the granular material behind the wall and help reduce clogging of the drainage system. In addition, weep holes should also be provided in the design of the walls. If a drainage system is not provided, the design should incorporate full hydrostatic forces behind the wall.

13. Based on total and differential settlement concerns, it is recommended that the CIP wall option be supported by deep foundation elements. The design of the deep foundation system should be based on the following toe and heel pressures calculated for the wall configurations described herein.

Applicable Bearing Pressures for Design of Deep Foundations for CIP Wall Option

	Wall Geometry**			Calculated Pressures		
Station Interval*	**Maximum Height**	**Footing Width**	**Toe Width**	**Meyerhof Uniform**	**Maximum Toe**	**Minimum Heel**
39+00 to 40+00	37.0 ft	0.8H	0.1H	4,880 psf	7,950 psf	3,520 psf
40+00 to 40+50	25.0 ft	0.8H	0.1H	4,670 psf	5,700 psf	2,330 psf
40+50 to 41+00	18.5 ft	0.8H	0.1H	3,680 psf	4,550 psf	1,680 psf
41+00 to 41+50	13.0 ft	1.0H	0.1H	2,740 psf	3,170 psf	1,830 psf
41+50 to 42+08	26.5 ft	0.8H	0.1H	4,910 psf	5,990 psf	2,480 psf

* Ramp 2 Stationing
** H = Wall Height

The bearing pressures provided above were determined based on the wall geometries outlined in the table. If the final design results in retaining wall geometries different than those outlined above, the retaining wall designer should perform analyses to determine the appropriate bearing pressures for design of the foundation system.

14. Axial capacity estimates for single steel H-piles are provided in Appendix I. The following table provides estimated pile lengths applicable for the recommended maximum total factored geotechnical axial resistances (TFGAR) along the wall alignment. Upon determination of the final H-pile locations, arrangement, and loads, the Designer should use the estimates to determine the H-pile size and length for each pile. However, the Designer should note that these estimates are for the TFGAR referenced in the following table only. Should more or less capacity be required, the Designer should consult FMSM because the downdrag load and length of pile subjected to downdrag are a function of the pile length.

Summary of Driven Pile Capacities

Maximum Total Factored Geotechnical Axial Resistance[a] (tons)	Depth[b] (ft)	Elevation (ft)	Total Factored Geotechnical Uplift Resistance[c] (tons)
12x53 H-pile			
100	70.0	398.1	82.9
14x73 H-pile			
140	71.5	396.6	116.3
14x89 H-pile			
170	74.5	393.6	139.4

a Excludes any positive resistance within downdrag zone.
b Depth as measured from the bottom of the pile cap.
c Reported uplift resistance is for the corresponding pile length.

15. The TFGAR estimates provided in Appendix I were derived using the following resistance factors, as recommended by the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

Loading Condition	Resistance Mechanism	Analysis Methodology	Resistance Factor* (ø)
Nominal Resistance of Single Pile in Axial Compression - Static Analysis	Skin Friction and End Bearing - Clay and Mixed Soils	α-Method	0.35
	Skin Friction and End Bearing - Sand	Nordlund/Thurman Method	0.45
Uplift Resistance of Single Piles - Static Analysis	Side Resistance in Clay	α-Method	0.25
	Side Resistance in Sand	Nordlund Method	0.35

* From AASHTO LRFD Bridge Design Specifications, Third Edition (including 2005 and 2006 Interim Revisions), portion of Table 10.5.5.2.3-1

16. If load testing and/or dynamic analysis of driven piles in soil is conducted, the LRFD resistance factors used to determine the factored axial capacity for design purposes may be revised as outlined in Table 10.5.5.2.3-1 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition based on site variability and the number and type of tests performed. If the Designer performed lateral capacity analyses based on the pile lengths outlined in Geotechnical Note 14, the lateral capacity analyses will need to be revisited if the pile lengths are revised based on load testing and/or dynamic analysis.

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) CIP RETAINING WALL

GEOTECHNICAL NOTES

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/.19

SHEET 2 OF 7

LX2004130/PHASE1/S9280/CIP/S9280_GEONTS1.DGN

GEOTECHNICAL NOTES

for CIP Cantilever Retaining Walls

17. Because the widening of the roadway embankment will be constructed after installation of the deep foundation elements recommended for support of the CIP wall, the piles will be subjected to downdrag loads resulting from settlement of the foundations soils. One of the following alternatives may be implemented to reduce the downdrag loads:

    a. Coat piles with bitumen slip layer within the zone subjected to downdrag to allow movement between the soil and the piles. Current practice allows for as much as 90 percent reduction in downdrag forces with this method.

    b. Predrill and provide a polypropylene or steel sleeve for the pile to reduce down-drag. This method only prevents contact between the pile and adjacent soils.

    c. Design the embankment with lightweight fill to reduce the overall settlement of the foundation soils.

18. As noted, all pile capacities presented in Appendix I are for single piles. In addition to applying appropriate resistance factors, individual capacities for piles in group configurations may be further reduced depending upon soil type, bearing condition of the pile cap, or center-to-center spacing as recommended in the AASHTO LRFD Bridge Design Specifications, Fourth Edition. The following criteria should be observed:

Group Efficiency Factor

	Cohesive Soils		Cohesionless Soils
CTC Spacing	Cap not in firm Contact with Ground	Cap in firm Contact with Ground	Cap in or not in firm Contact with Ground
6B	1.00	1.00	1.00
2.5B	0.65	1.00	1.00

The notation 'B' is the pile diameter and the percent reduction can be linearly interpolated between the values and spacing provided.

19. The AASHTO LRFD Bridge Design Specifications recommend a resistance factor for horizontal geotechnical resistance of a single pile or pile group of 1.0 for lateral capacity analyses. Appendix H provides Idealized Soil Profiles that outline the recommended soil parameters for use in lateral load analyses.

20. Use Grade 50 steel H-piles as friction piles. Piles should be driven to the target elevation and then left for a minimum of one day to allow for dissipation of excess pore pressures caused by the pile installation process. This should allow the soil to 'set-up'. After the one day waiting period, re-strike the piles to see if an adequate capacity has been achieved.

21. Hammer energies which could drive the pile section were based on the ultimate driving resistance that 12x53, 14x73 and 14x89 steel H-piles would experience during the installation process. The results of these calculations are presented in the following table:

Summary of Driven Pile Capacities

Maximum Total Factored Geotechnical Axial Resistance (tons)	Depth[a] (ft)	Tip Elevation[b] (ft)
12x53 H-pile		
23	70.0[c]	398.1[c]
40	70.0[c]	398.1[c]
60	70.0[c]	398.1[c]
14x73 H-pile		
23	65.5	402.6
40	71.5[c]	396.6[c]
60	71.5[c]	396.6[c]
14x89 H-pile		
23	65.0	403.1
40	73.5	394.6
60	74.5[c]	393.6[c]

a Depth as measured from the bottom of the pile cap.
b Based upon estimated bottom of pile cap at elevation 468.1 feet.
c Depth/Elevation corresponding to the maximum TFGAR

22. Upon selecting the pile size and length required to support the applied loads, the Designer should select the minimum hammer energy required to drive the piles to the specified depths from the table presented in Geotechnical Note 21 above. The Designer should place a note on the drawings that states: A hammer system capable of delivering a minimum energy of --- foot-kips will be necessary to drive the piles without encountering excessive blow counts and over-stressing the piles. The Contractor should submit appropriate pile driving systems to the Kentucky Transportation Cabinet for approval prior to the installation of the first pile. Approval of the pile driving system by the Engineer will be subject to satisfactory field performance of the pile driving procedures.

23. Upon selecting the pile size and length required to support the applied loads, the Designer should select the minimum driving resistance required to install the pile to the design depth from the tables provided in Appendix J. This driving resistance should be reported to the Contractor to aid in determining when/if the pile has been driven to a sufficient depth to achieve the specified capacity.

24. Pile types, driving systems and installations should conform to current AASHTO LRFD Bridge Design Specifications unless otherwise specified.

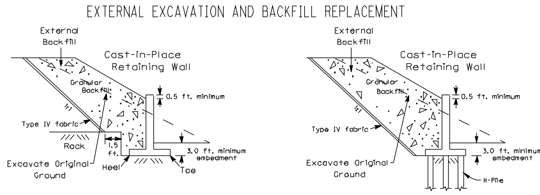
25. Drivability studies were performed assuming continuous driving. If interruptions in driving individual piles should occur, difficulties in continuing the installation process will likely occur due to pile 'set-up' characteristics.

26. The AASHTO LRFD Bridge Design Specifications, Fourth Edition recommends the following resistance factors for determining the structural capacity of steel H-piles.

	Resistance Factor*	
Loading Condition	Piles Subjected to Damage from severe Driving Conditions	Good Driving Conditions
Axial Resistance in Compression	$\phi_c = 0.50$	$\phi_c = 0.60$
Combined Axial and Flexural Resistance	N/A	$\phi_c = 0.70$ $\phi_f = 1.00$

* As specified in Section 6.5.4.2 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition
**Apply these values only to the section of the pile likely to be damaged during driving (Section 6.15.2 of the AASHTO Specifications)

EXTERNAL EXCAVATION AND BACKFILL REPLACEMENT



Use the following soil strength parameters for design:

	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
Retained Fill			
Soil Embankment	170	27	120
Granular Embankment	0	38	120
Foundation Soils			
Existing Clay Soils	150	32	128
Existing Embankment	0	35	122
Granular Replacement	0	38	120

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-65 | CROSSING S9280 (W65-10) CIP RETAINING WALL

GEOTECHNICAL NOTES

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/.19

LX2004130/PHASE1/S9280/CIP/S9280_GEONTS2.DGN

SHEET LOCATION:
FILE NAME: $$$$design$file$specification$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

GEOTECHNICAL NOTES

for Toe Wall

1. Design of the subject retaining structure should be in accordance with the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

2. Based on the subsurface conditions encountered in borings performed along the wall alignment, the wall will be a soil bearing structure. It is recommended that the minimum wall embedment depth be in accordance with Section 11 of the AASHTO LRFD Bridge Design Specifications, Fourth Edition.

3. Review of the soil profile developed along the wall alignment in conjunction with the planned bearing elevations indicate the wall will be founded on clayey soils from Station 39+00 to 40+00 and shale (shot rock) embankment from Station 40+00 to 42+08. Thus, the bearing capacity will be controlled by the short-term strength of the clays and the friction angle of the shale. Based on a value of cohesion of 790 psf and a friction angle of 35 degrees for the alluvial clays and shale (shot rock) embankment materials, respectively, derived from unconfined compression test results and correlations of corrected SP N-values, the estimated nominal bearing capacity (qn) of the soils between Station 39+00 and 40+00 is on the order of 4,226 psf. The estimated nominal bearing capacity (qn) of the shale embankment between Station 40+00 and 42+08 ranges from 27,050 psf to 44,308 psf.

4. Construction of the planned retaining wall will require excavations at the toe of the existing interstate and ramp embankments as well as temporary excavations within the embankments themselves. The Contractor should evaluate the stability of the existing embankments and adjacent structures in conjunction with temporary excavations to verify that the planned excavation/bracing/shoring system maintains the stability of the highway embankment.

5. Temporary wall slopes and foundation excavations in soil shall be properly designed, or should be properly braced/shored to reduce the possibility of collapse and provide adequate safety to people working in or around the excavations. Bracing/shoring shall be performed in accordance with applicable federal, state and local guidelines.

6. At the writing of this report, a borrow source for embankment material has not been identified. The engineering analyses performed for the retaining wall options are based on estimated soil strength parameters for the retained fill and embankment materials. It is recommended that borrow material to be used for embankment construction meet the following minimum strength parameters.

Embankment Material		Retained Fill	
Total Stress	**Effective Stress**	**Total Stress**	**Effective Stress**
c = 1400 psf	$\bar{c}$ = 200 psf	c = 1400 psf	$\bar{c}$ = 170 psf
ϕ = 0°	$\bar{\phi}$ = 23°	ϕ = 0°	$\bar{\phi}$ = 27°
γ = 120 pcf	γ = 120 pcf	γ = 120 pcf	γ = 120 pcf

The retained fill material shall be placed in the entire area between the wall and a 1:1 (H:V) line sloping upward and away from the base of the heel of the wall for a CIP wall or the base of the reinforced material for a MSE wall to the top of the wall. Non-durable shales and fat clays (USCS classification of CH) should specifically be excluded from use within this zone.

The Contractor shall perform laboratory testing to confirm that the minimum total stress and effective stress strength parameters are equal to or greater than the above values per material type for each borrow area. The test results shall be submitted to the Engineer for approval.

7. Fill materials associated with interstate construction and/or previous development in the City of Louisville were encountered within each of the five borings drilled along the wall alignment. Because the structure site is located within an area of previous site grading and construction, the Contractor should anticipate encountering fill materials along the wall alignment. The excavations shall be deepened as necessary to provide an adequate bearing medium.

8. Granular embankment used as backfill and/or for over excavation and replacement shall be non-erodible and shall conform to the requirements of Section 805 of the current Kentucky Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The granular embankment material shall be wrapped with Type IV geotextile fabric meeting the requirements of Section 843 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction to provide separation from the clay embankment and/or foundation materials.

9. Soils exposed within the bottoms of footing trenches shall be observed for suitability by a geotechnical engineer or an engineering technician working under his/her direct supervision. Old fill or unsuitable material which might be encountered shall be removed. Areas disturbed by the excavation process should be restored utilizing proper compaction methods.

10. If soft, wet soils are encountered in the foundation excavations, they should be undercut a minimum depth of two (2) feet and backfilled to the design bearing elevation using non-erodible Granular Embankment conforming to Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Contrary to Section 805 of the specifications, the maximum size limit shall be reduced to four (4) inches. The bottoms of the foundation excavations should be proof-rolled to restore the in-place density of any soil disturbed in the excavation process. Isolated zones of loose or wet soil may also be stabilized using KY size No. 2, 3, or 23 stone, as specified in Section 805 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

11. Based on the depth to bedrock and the anticipated wall loading, it is recommended that the retaining wall be supported by a yielding foundation system. The allowable bearing capacity of the underlying soil material is 2,230 pounds per square foot.

12. To maintain an acceptable factor for the CDRsliding, it is recommended that a minimum of one foot of material be excavated below the wall footprint and backfilled with non-erodible granular embankment as defined in Geotechnical Note 8.

13. The minimum wall embedment shall be two feet, as measured from the ground surface in front of the wall down to the base of the footing.

14. Retaining wall stability analyses indicate the geometry for a six foot tall standard gravity wall (KYTC Standard Drawing RGX-002-07) will meet the minimum factor for CDRsliding based on the AASHTO LRFD Bridge Design Specifications, Fourth Edition. If a gravity type retaining wall is chosen, the foundation soils will need to be over excavated and replaced with granular embankment (see Geotechnical Note 12), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Geotechnical Note 8. As another option, a gravity type retaining wall can be used with random backfill provided that the foundation soils are over excavated and replaced with granular embankment and the base width is widened to 7.2 feet.

Using a phi angle of 38 degrees, a wall height measuring 6 feet, a base width measuring 3 feet, an angle between back of the wall and vertical equal to 14.0 degrees, a friction angle between the back of the wall and the granular backfill equal to 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	41 psf
3:1(H:V)	53 psf
2:1(H:V)	64 psf

The backfill shall be placed in the entire area between the wall and a 1:1(H:V) line sloping upward away from the base of the heel of the wall to the top of the wall. Type IV geotextile fabric shall be placed on the 1:1(H:V) slope to reduce the infiltration of fines into the granular material behind the wall and help prevent clogging of the drainage system. The drainage system shall consist of 4 inch diameter pipe with weepholes installed at locations as indicated by KYTC Standard Drawing RGX 002-07 or by the Designer; and/or perforated pipe installed at the base of the wall and 'daylighted' to promote 'dewatering' of the granular backfill.

15. Retaining wall stability analyses indicate a cantilever-type retaining wall measuring six feet in height can meet the minimum factor for CDRsliding and eccentricity provided that the foundation soils are over excavated and replaced with granular embankment (see Geotechnical Note 12). In addition, the footing width will need to be increased to 4.8 feet (0.8H), and the backfill behind the wall shall consist of non-erodible granular embankment as defined in Geotechnical Note 8.

Using a phi angle of 38 degrees, a vertical back of wall, friction angle between the back of the wall and backfill of 29 degrees, and a unit weight of 120 pounds per cubic foot for the backfill material, the following equivalent fluid pressures are applicable:

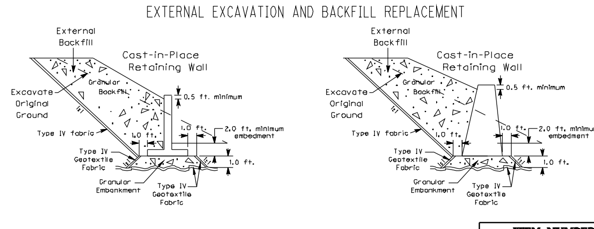
Slope of Backfill	Equivalent Fluid Pressure Per Linear Foot Granular Embankment (ϕ = 38°)
Level	26 psf
3:1(H:V)	33 psf
2:1(H:V)	39 psf

Drainage systems consisting of free draining material and filter fabric shall be placed directly behind the wall and be a minimum thickness of two (2) feet. Use of filter fabric will help reduce the infiltration of fines into the granular material behind the wall and help reduce clogging of the drainage system. In addition, weep holes shall also be provided in the design of the walls. If a drainage system is not provided, the design shall incorporate full hydrostatic forces behind the wall.

Use the following soil strength parameters for design:

	Cohesion (psf)	Friction Angle (degrees)	Unit Weight (pcf)
External Backfill			
Soil Embankment	200	23	120
Granular Embankment	0	38	120
Foundation Soils			
Existing	150	32	128
Granular Replacement	0	38	120

EXTERNAL EXCAVATION AND BACKFILL REPLACEMENT



ITEM NUMBER
5-118.18/19

SHEET 4 OF 7

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

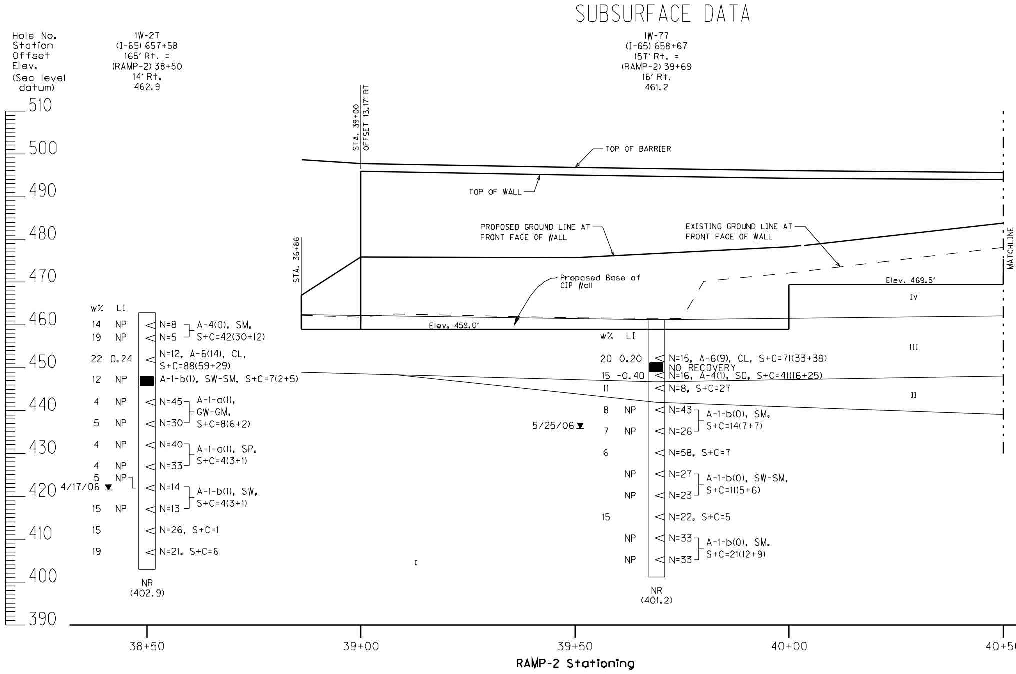
ROUTE I-65 | CROSSING S9280 (W65-10) TOE WALL

GEOTECHNICAL NOTES

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

SUBSURFACE DATA



For clarity, the planned toe wall is not shown in the elevation view depicted on this sheet.

SUMMARY OF ESTIMATED SETTLEMENT PARAMETERS	
SOIL LAYER	SETTLEMENT PARAMETERS
IV	C' = 65 γ = 122 pcf
III	e_o = 0.765 C_c = 0.198 C_r = 0.033 P_c = 1,000 psf γ = 128 pcf
II	C' = 39 γ = 113 pcf
I	C' = 55 γ = 124 pcf

NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

NOTE:
Over excavation not applicable for the cast-in-place wall option because deep foundation elements are recommended to support the proposed retaining structure.

REVISION	DATE
DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-65 | CROSSING S9280 (W65-10) CIP RETAINING WALL

BORING LOGS AND PROFILE

PREPARED BY
BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

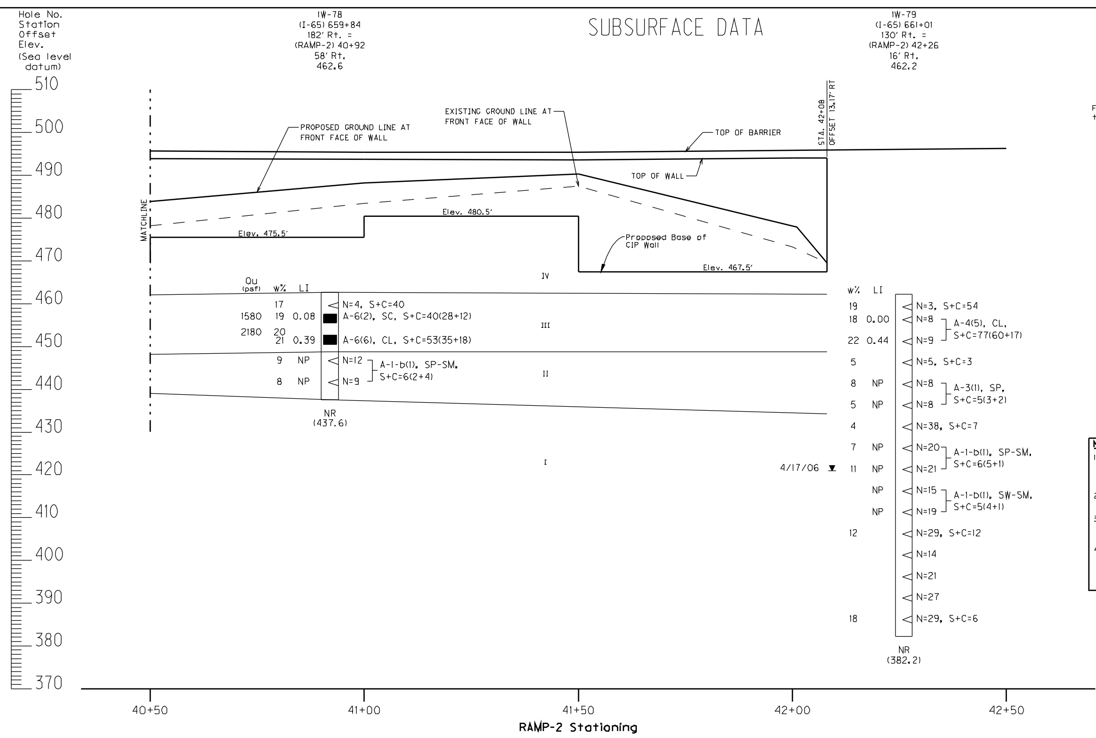
SCALE: 1" = 10'

ITEM NUMBER
5-118.18/19

LX2004130/PHASE1/S9280/CIP/S9280_PROF1.DGN

SHEET LOCATION:
FILE NAME: $$$$designfilespecification$$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

SUBSURFACE DATA



For clarity, the planned toe wall is not shown in the elevation view depicted on this sheet.

SUMMARY OF ESTIMATED SETTLEMENT PARAMETERS	
SOIL LAYER	SETTLEMENT PARAMETERS
IV	C' = 65 γ = 122 pcf
III	e_o = 0.765 C_c = 0.198 C_r = 0.033 P_c = 1,000 psf γ = 128 pcf
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NOTES:

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2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall elevation used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

NOTE:
Over excavation not applicable for the cast-in-place wall option because deep foundation elements are recommended to support the proposed retaining structure.

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PVK, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE: I-65
CROSSING: S9280 (W65-10) CIP RETAINING WALL

BORING LOGS AND PROFILE

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/.19

SHEET 6 OF 7

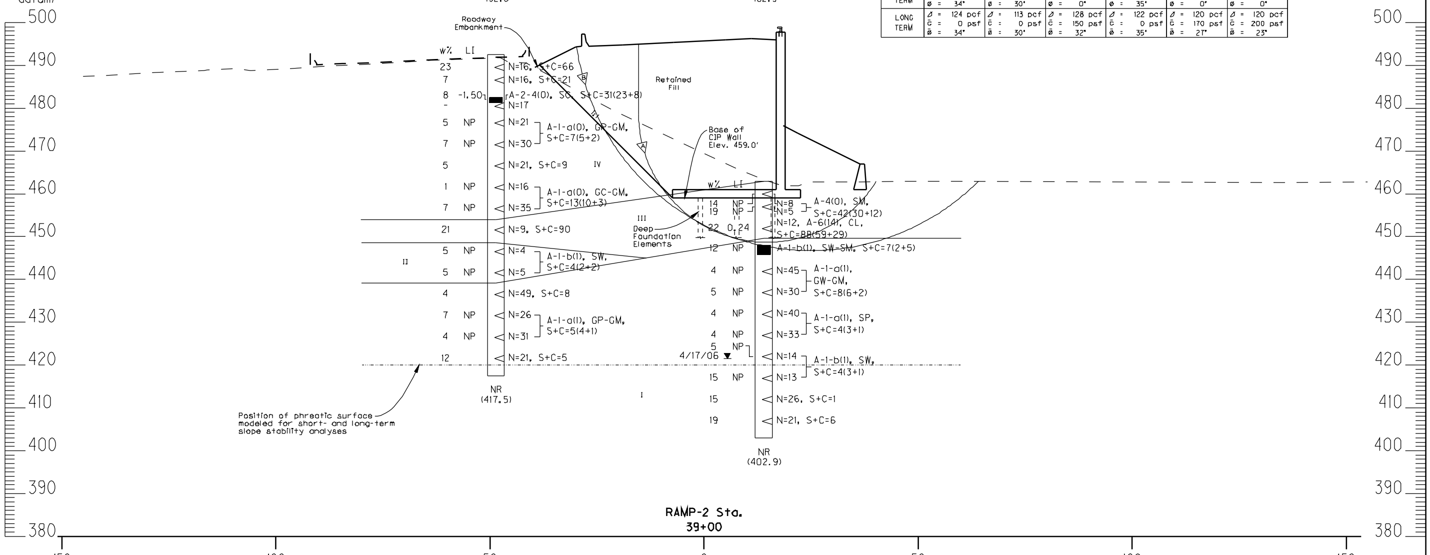
LX2004130/PHASE1/S9280/CIP/S9280_PROF2.DGN

SHEET LOCATION:
FILE NAME: $$$$designfilespecification$$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

SUBSURFACE DATA

FACTORS OF SAFETY		
SHORT TERM	A	1.3
LONG TERM	B	2.4

ESTIMATED SOIL STRENGTH PARAMETERS						
SOIL	I	II	III	IV	RETAINED FILL	ROADWAY EMBANKMENT
SHORT TERM	γ = 124 pcf c = 0 psf ø = 34°	γ = 113 pcf c = 0 psf ø = 30°	γ = 128 pcf c = 790 psf ø = 0°	γ = 122 pcf c = 0 psf ø = 35°	γ = 120 pcf c = 1400 psf ø = 0°	γ = 120 pcf c = 1400 psf ø = 0°
LONG TERM	γ = 124 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 34°	γ = 113 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 30°	γ = 128 pcf $\bar{c}$ = 150 psf $\bar{ø}$ = 32°	γ = 122 pcf $\bar{c}$ = 0 psf $\bar{ø}$ = 35°	γ = 120 pcf $\bar{c}$ = 170 psf $\bar{ø}$ = 27°	γ = 120 pcf $\bar{c}$ = 200 psf $\bar{ø}$ = 23°



NOTES:

1. This sheet presents geotechnical data and recommendations. Refer to project plans, profiles, and cross sections for final alignment and grade.
2. Surface elevations are referenced to Mean Sea Level.
3. All standard penetration testing performed for structure borings were done utilizing an automatic hammer.
4. The information for the retaining wall cross-section used on this drawing was obtained through the Bentley Projectwise Program from the Kentucky Transportation Associates Design Team on June 12, 2006.

SCALE: 1" = 10'

REVISION	DATE

DATE: APRIL 2007	CHECKED BY
DESIGNED BY:	
DETAILED BY: PJB	PKV, DLB

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE	CROSSING
I-65	S9280 (W65-10) CIP RETAINING WALL

STABILITY SECTION

PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

SHEET NO.
DRAWING NO.

ITEM NUMBER
5-118.18/19

SHEET 7 OF 7

LX2004130/PHASE1/S9280/CIP/S9280_STAB.DGN

SHEET LOCATION:
FILE NAME: $$$$designfilespecification$$$$
USERNAME: $$$$USER$$$$
DATE: $$$$DATE$$$$
E-SHEET NAME:

Appendix E

Coordinate Data Submission Form

COORDINATE DATA SUBMISSION FORM
KYTC DIVISION OF MATERIALS - GEOTECHNICAL BRANCH

County: Jefferson
Road Number: Ohio River Bridges - Phase 1
Survey Crew / Consultant: Qk4, Inc.
Contact Person: Jim Krauth
Item No.: 5-118.18/ .19
Mars No.: C-04224166
Project No.: N/A

Date: December 30, 2005

Notes: S9280 (W65-10) extends along the east side of I-65 from Jefferson Street to Market Street.

(select one) **Elevation Datum** **Sea Level** **Assumed**

HOLE NUMBER	STATION	OFFSET	ELEVATION (ft)	LATITUDE	LONGITUDE
1W-27	657+58	165' Rt.	462.9	N 38.252830632°	W 85.743698133°
1W-77	658+67	157' Rt.	461.2	N 38.253051128°	W 85.743381588°
1W-78	659+84	182' Rt.	462.6	N 38.253239199°	W 85.742986423°
1W-368	659+85	114' Rt.	492.5	N 38.253376746°	W 85.743149263°
1W-79	661+01	130' Rt.	462.2	N 38.253598137°	W 85.742810041°

Appendix F

Laboratory Testing Results

Reconstructed by C. Barnett 6/20/06
Checked by WJC 6/20/06

FMSM ENGINEERS
Fuller Mossbarger Scott & May

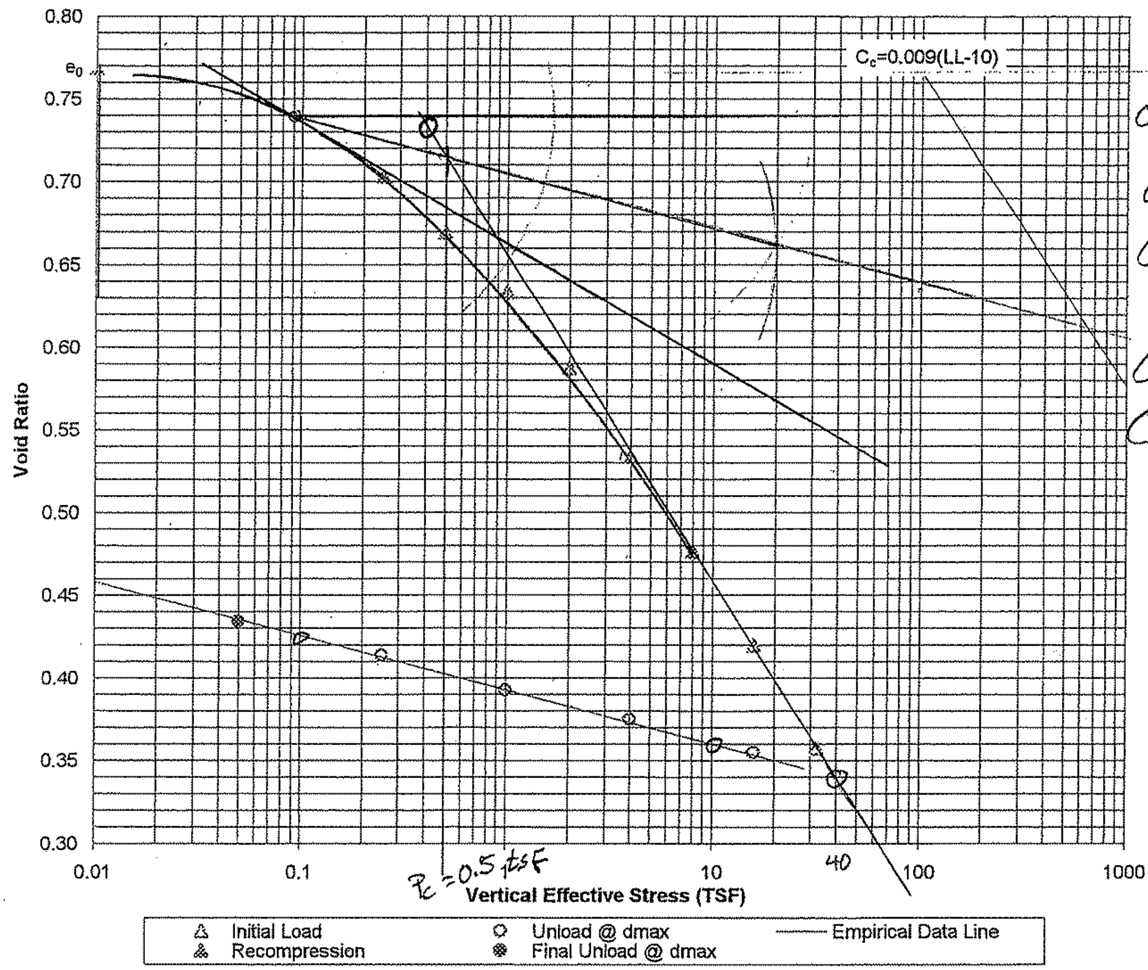
One-Dimensional Consolidation of Soils

ASTM D 2435

Project Name	Kennedy Interchange Phase 1	Project No.	LX2004130
Source	1W-78/659+84, 182' Rt., 5.0' - 7.0', Tl 6.8' - 7.0'	Sample ID	1016
Cv computation Method: Square Root of Time		Initial Void Ratio	0.765
Swell Pressure (tsf) - 5.24 E-02		Preconsolidation Pressure (tsf)	

$C_c = 0.198$
$C_r = 0.033$

Void Ratio at d_{100} vs. Stress



$C_c = \frac{0.735-0.31}{\log\left(\frac{40}{0.4}\right)}$

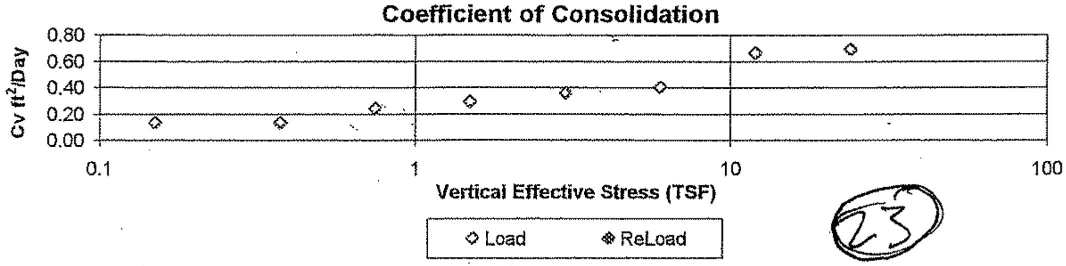
$C_c = 0.198$

$C_r = \frac{0.426-0.361}{\log\left(\frac{10}{0.1}\right)}$

$C_r = 0.033$

$C_v = 0.22$

Coefficient of Consolidation



23

KDG

Appendix G

Correction of SPT Data

KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
Soil No.	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_w	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e

NOTES:

C. This spreadsheet has been designed such that an initial "Assumed Estimated Unit Weight" is placed into Column C.

E. N_{80} is the blow count per foot as determined in the field using a automatic hammer.

F. $N_{60} = (E_{AH}/60)\, N_{AH}$, where: E_{AH} = autohammer efficiency (80%): N_{AH} = blowcount from the autohammer, as referenced in (1)
The autohammer efficiency is based on typical values of efficiencies (85 - 95) and actual testing preformed on FMSM hammers. SPT Analyzer equipment from Pile Dynamics Inc. was used to conduct the testing. An autohammer is more energy efficient than a standard hammer.
Hammer efficiency is a means of comparing the energy transferred from the hammer to the drill string during sampling.

G. Correction Factor Based on 1/(square root of vertical effective stress). (Liao, S.C. and Whitman, R.V. 1985.
"Overburden Correction Factors for SPT in Sand", JGED, ASCE, Vol. 112, No. 3, pp. 373-377; as referenced in (2).
This correction factor is limited to vertical effective stresses greater than 0.25 tsf.

I. Relative Density based on Tokimatsu, K. and Seed, H.B. 1988. "Evaluation of Settlements in Sands Due to Earthquake Shaking",
JGED, ASCE, Vol. 113, No. 8, pp. 861-878; as referenced in (2).

J. Classification based on field and laboratory data by FMSM.

K, L and O Angle of Internal Friction (phi), Unit Weight Dry and Void Ratio based on NAVFAC 7.1 "Soil Mechanics", May 1982, page 7.1-149.

M. Moisture content based on laboratory testing of SPT samples by FMSM.

N. In-situ unit weight is based on dry unit weight (L) times (1 + moisture content).

(1) Goble, George, GRL Newsletter, December 1995 "SPT Improvements"

(2) Seed and Harder, Volume 2 Memorial Symposium Proceedings, May 1990. "SPT Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength", pp. 361-362.

KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
		γ_t	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
				Input Required										
1W-27		water =	41.4											
2.0 - 3.5	2.75	115	0.16	8	11	1.00	11	47	SM	31	96	13.9	109	0.74
5.0 - 6.5	5.75	115	0.33	5	7	1.00	7	35	SM	30	94	19.0	112	0.78
10.0 - 11.5	10.75	115	0.62	12	16	1.00	16	60	CL	NA	NA	21.8	NA	NA
15.0 - 17.0	16	115	0.92	ST	NA	1.00	NA	N/A	SW-SM	N/A	N/A	NA	N/A	N/A
20.0 - 21.5	20.75	115	1.19	45	60	0.92	55	99	GW-GM	41	145	3.6	150	NA
25.0 - 26.5	25.75	115	1.48	30	40	0.82	33	82	GW-GM	41	140	5.3	147	NA
30.0 - 31.5	30.75	115	1.77	40	53	0.75	40	89	SP	38.5	120.5	3.9	125	0.39
35.0 - 36.5	35.75	115	2.06	33	44	0.70	31	79	SP	37	117.5	3.5	122	0.42
40.0 - 41.5	40.75	115	2.34	14	19	0.65	12	52	SW	33.6	112	5.2	118	0.49
45.0 - 46.5	45.75	115	2.47	13	17	0.64	11	47	SW	33	111	14.6	127	0.5
50.0 - 51.5	50.75	115	2.61	26	35	0.62	22	68	SW	35.5	115	15.4	133	0.45
55.0 - 56.5	55.75	115	2.74	21	28	0.60	17	60	SW	35	114	19.4	136	0.46

KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval		Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
			γ_t	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
1W-77			water =	25.4	5/25/2006										
8.0 -	9.5	8.75	115	0.50	15	20	1.00	20	67	CL	NA	NA	19.6	NA	NA
10.0 -	12.0	11.00	115	0.63	ST	NA	1.00	NA	N/A	NO RECOVERY	NA	NA	NA	NA	NA
12.0 -	13.5	12.75	115	0.73	16	21	1.00	21	68	SC	33.5	100	14.9	115	0.67
15.0 -	16.5	15.75	115	0.91	8	11	1.00	11	47	SC	31	96	11.3	107	0.74
20.0 -	21.5	20.75	115	1.19	33	44	0.92	40	89	SM	36	105	7.6	113	0.6
25.0 -	26.5	25.75	115	1.32	26	35	0.87	30	79	SM	34.5	102	6.8	109	0.63
30.0 -	31.5	30.75	115	1.46	58	77	0.83	64	100	SW-SM	39	116	5.5	122	0.44
35.0 -	36.5	35.75	115	1.59	27	36	0.79	29	77	SW-SM	36	110	6.0	117	0.52
40.0 -	41.5	40.75	115	1.72	23	31	0.76	23	71	SW-SM	35	109	15.0	125	0.54
45.0 -	46.5	45.75	115	1.85	22	29	0.74	22	68	SW-SM	34.5	108	15.4	125	0.55
50.0 -	51.5	50.75	115	1.98	33	44	0.71	31	81	SM	35.5	104	15.0	120	0.61
55.0 -	56.5	55.75	115	2.11	33	44	0.69	30	79	SM	34.5	102	15.0	117	0.63

KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval		Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
			γ_t	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
1W-78			water =	40.0											
2.0 -	3.5	2.75	115	0.16	4	5	1.00	5	32	SC	29.5	93	16.9	109	0.8
5.0 -	7.0	6	115	0.35	ST	NA	1.00	NA	N/A	SC	N/A	N/A	18.7	N/A	N/A
10.0 -	12.0	11	115	0.63	ST	NA	1.00	NA	N/A	CL	NA	NA	20.8	NA	NA
15.0 -	16.5	15.75	115	0.91	12	16	1.00	16	60	SP-SM	34	107	8.5	116	0.56
20.0 -	21.5	20.75	115	1.19	9	12	0.92	11	47	SP-SM	32	104	7.9	112	0.61

KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval		Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf)	Vertical Effective Stress (tsf)	SPT N Value	SPT N Value	Correction Factor	Corrected N-Value	Relative Density (%)	Unified Soil Classification	Internal Angle of Friction (degrees)	Unit Weight Dry (pcf)	Moisture Content (%)	Revised In-situ Unit Weight (pcf)	Void Ratio
			γ_k	σ'	N_{80}	N_{60}	C_N	$(N_1)_{60}$	D_r		ϕ'	γ_d	m	γ_w	e
					Input Required										
1W-79			water =	41.3											
2.0 -	3.5	2.75	115	0.16	3	4	1.00	4	27	CL	NA	NA	18.6	NA	NA
5.0 -	6.5	5.75	115	0.33	8	11	1.00	11	47	CL	NA	NA	17.5	NA	NA
10.0 -	11.5	10.75	115	0.62	9	12	1.00	12	52	CL	NA	NA	22.1	NA	NA
15.0 -	16.5	15.75	115	0.91	5	7	1.00	7	35	SP	31.5	109	5.3	115	0.54
20.0 -	21.5	20.75	115	1.19	8	11	0.92	10	44	SP	32.3	110	7.8	119	0.52
25.0 -	26.5	25.75	115	1.48	8	11	0.82	9	41	SP	32.3	110	5.0	116	0.52
30.0 -	31.5	30.75	115	1.77	38	51	0.75	38	87	SP	38.5	120.5	3.7	125	0.39
35.0 -	36.5	35.75	115	2.06	20	27	0.70	19	63	SP-SM	34	107	6.9	114	0.56
40.0 -	41.5	40.75	115	2.34	21	28	0.65	18	63	SP-SM	34	107	11.2	119	0.56
45.0 -	46.5	45.75	115	2.47	15	20	0.64	13	52	SW-SM	33	105	11.0	117	0.59
50.0 -	51.5	50.75	115	2.61	19	25	0.62	16	58	SW-SM	33.5	106	11.0	118	0.57
55.0 -	56.5	55.75	115	2.74	29	39	0.60	23	71	SW-SM	35	109	12.4	123	0.54
60.0 -	61.5	60.75	115	2.87	14	19	0.59	11	47	SW-SM	32	104	12.0	116	0.61
65.0 -	66.5	65.75	115	3.00	21	28	0.58	16	60	SW-SM	34	107	18.0	126	0.56
70.0 -	71.5	70.75	115	3.13	27	36	0.57	20	67	SW-SM	34.5	108	18.0	127	0.55
75.0 -	76.5	75.75	115	3.26	29	39	0.55	21	68	SW-SM	34.5	108	18.4	128	0.55

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KENNEDY INTERCHANGE
CORRELATION OF SPT DATA TO UNIT WEIGHTS AND SHEAR STRENGTHS
FOR COARSE GRAINED SOILS

Sample Interval	Depth of Mid. Pt. of Sample (ft.)	Assumed Estimated Unit Weight (pcf) γ_e	Vertical Effective Stress (tsf) σ'	SPT N Value N_{80}	SPT N Value N_{60}	Correction Factor C_N	Corrected N-Value $(N_1)_{60}$	Relative Density (%) D_r	Unified Soil Classification	Internal Angle of Friction (degrees) ϕ'	Unit Weight Dry (pcf) γ_d	Moisture Content (%) m	Revised In-situ Unit Weight (pcf) γ_w	Void Ratio e
				Input Required										
1W-368		water =	Dry	4/4/2006										
2.0 - 3.5	2.75	115	0.16	16	21	1.00	21	68	CL	NA	NA	22.7	NA	NA
5.0 - 6.5	5.75	115	0.33	16	21	1.00	21	68	CL	NA	NA	7.0	NA	NA
10.0 - 11.0	10.50	115	0.60	ST	NA	1.00	NA	N/A	SC	N/A	N/A	8.4	N/A	N/A
11.0 - 12.5	11.75	115	0.68	17	23	1.00	23	70	GP-GM	37	121	5.4	128	0.38
15.0 - 16.5	15.75	115	0.91	21	28	1.00	28	77	GP-GM	37.6	122	6.6	130	0.37
20.0 - 21.5	20.75	115	1.19	30	40	0.92	37	86	GP-GM	39.3	125	6.0	133	0.34
25.0 - 26.5	25.75	115	1.48	21	28	0.82	23	71	GC-GM	37	121	4.8	127	0.38
30.0 - 31.5	30.75	115	1.77	16	21	0.75	16	60	GC-GM	35.5	118.5	1.4	120	0.41
35.0 - 36.5	35.75	115	2.06	35	47	0.70	33	82	GC-GM	38.5	123	6.9	131	0.35
40.0 - 41.5	40.75	115	2.34	9	12	0.65	8	41	SW	32.3	110	21.4	134	0.52
45.0 - 46.5	45.75	115	2.63	4	5	0.62	3	24	SW	29.5	106	5.2	112	0.58
50.0 - 51.5	50.75	115	2.92	5	7	0.59	4	24	SW	29.5	106	5.2	112	0.58
55.0 - 56.5	55.75	115	3.21	49	65	0.56	37	86	GP-GM	39.3	125	3.5	129	0.34
60.0 - 61.5	60.75	115	3.49	26	35	0.54	19	63	GP-GM	35.5	118.5	6.7	126	0.41
65.0 - 66.5	65.75	115	3.78	31	41	0.51	21	68	GP-GM	36.2	120	3.6	124	0.39
70.0 - 71.5	70.75	115	4.07	21	28	0.50	14	53	GP-GM	34.2	116	11.8	130	0.44

Appendix H

Idealized Soil Profiles

SOIL PROFILE LEGEND SHEET

Kennedy Interchange
Retaining Wall S9280 (W65-10)

SUMMARY OF PARAMETERS DEVELOPED FOR SOIL AND BEDROCK PROFILES

Parameter	Units	Description and Reference
γ_t	lb/ft^3	Total Unit Weight
γ_e	lb/ft^3	Effective Unit Weight
q_u	ton/ft^2	Uniaxial Compressive Strength (either soil or rock)
c_u	ton/ft^2	Undrained Shear Strength (either soil or rock)
ϕ	(°)	Angle of Internal Friction
k_s	lb/in^3 (soil)	Secant Modulus {computer program LPILEPLUS}
E_{50}	lb/in^2	Strain, {Value of strain at 50% of the maximum stress}

SOIL PROFILE

Kennedy Interchange
Retaining Wall S9280 (W65-10)
Ramp 2 Stations 39+00 to 40+00
(Based on Hole Nos. 1W-27 and 1W-77)

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description	Parameters	
462.9 - 461.2	0.0			
		Lean Clay with Silt and Sand (CL, SM, and SC)	γ_t (lb/ft^3) = 128.0 c_u (tsf) = 0.40 k_s (lb/in^3) = 100.0 E_{50} = 0.007	
				P-Y Curve Reference Number 3
449.0 - 447.5	13.9 - 13.7			
		Sand with Gravel (SP, and SP-SM)	γ_t (lb/ft^3) = 113.0 ϕ (^{o}) = 30.0 k_s (lb/in^3) = 25.0	
1W-77 Only				P-Y Curve Reference Number 4
441.5	19.7			
420.0	▽	Sand with Silt and Gravel (SM, SP-SM, SW-SM, and SW)	γ_t (lb/ft^3) = 124.0 γ_e (lb/ft^3)* = 61.6 ϕ (^{o}) = 34.0 k_s (lb/in^3) = 90.0 k_s (lb/in^3) = 60.0	 (above water table) (below water table)
				P-Y Curve Reference Number 4
335.6*	127.3 - 125.6	Top of Rock		

* Based on Refusal in Hole 1B-32

Note: A range in elevation and depths are being provided because of the variance between applicable borings

SOIL PROFILE

Kennedy Interchange
Retaining Wall S9280 (W65-10)
Ramp 2 Stations 40+00 to 40+50
(Based on Hole Nos. 1W-77, 1W-368 and 1W-78)

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description	Parameters	
470.5	0.0			
		Gravel with Sand (Shale Shot Rock) (SC, GP-GM, and GC-GM)	γ_t (lb/ft^3) = 122.0 ϕ (^{o}) = 35.0 k_s (lb/in^3) = 90.0	
				P-Y Curve Reference Number 4
461.5 - 462.6	9.0 - 7.9			
		Lean Clay with Silt and Sand (CL, SM, and SC)	γ_t (lb/ft^3) = 128.0 c_u (tsf) = 0.40 k_s (lb/in^3) = 100.0 E_{50} = 0.007	
				P-Y Curve Reference Number 3
447.5 - 449.5	23.0 - 21.0			
		Sand with Gravel (SP, and SP-SM)	γ_t (lb/ft^3) = 113.0 ϕ (^{o}) = 30.0 k_s (lb/in^3) = 25.0	
				P-Y Curve Reference Number 4
441.5 - 440.5	29.0 - 30.0			
420.0	▽	Sand with Silt and Gravel (SM, SP-SM, SW-SM, and SW)	γ_t (lb/ft^3) = 124.0 γ_e (lb/ft^3)* = 61.6 ϕ (^{o}) = 34.0 k_s (lb/in^3) = 90.0 k_s (lb/in^3) = 60.0	 (above water table) (below water table)
				P-Y Curve Reference Number 4
335.6*	134.9	Top of Rock		

* Based on Refusal in Hole 1B-32

Note: A range in elevation and depths are being provided because of the variance between applicable borings

SOIL PROFILE

Kennedy Interchange
Retaining Wall S9280 (W65-10)
Ramp 2 Stations 40+50 to 41+00
(Based on Hole Nos. 1W-368 and 1W-78)

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description	Parameters	
476.5	0.0			
		Gravel with Sand (Shale Shot Rock) (SC, GP-GM, and GC-GM)	γ_t (lb/ft^3) = 122.0 ϕ (°) = 35.0 k_s (lb/in^3) = 90.0	
				P-Y Curve Reference Number 4
462.5 - 462.5	14.0 - 14.0			
		Lean Clay with Silt and Sand (CL, SM, and SC)	γ_t (lb/ft^3) = 128.0 c_u (tsf) = 0.40 k_s (lb/in^3) = 100.0 E_{50} = 0.007	
				P-Y Curve Reference Number 3
449.5 - 448.5	27.0 - 28.0			
		Sand with Gravel (SP, and SP-SM)	γ_t (lb/ft^3) = 113.0 ϕ (°) = 30.0 k_s (lb/in^3) = 25.0	
				P-Y Curve Reference Number 4
440.5 - 437.5	36.0 - 39.0			
420.0	▽	Sand with Silt and Gravel (SM, SP-SM, SW-SM, and SW)	γ_t (lb/ft^3) = 124.0 γ_e (lb/ft^3)* = 61.6 ϕ (°) = 34.0 k_s (lb/in^3) = 90.0 (above water table) k_s (lb/in^3) = 60.0 (below water table)	
				P-Y Curve Reference Number 4
335.6*	140.9	Top of Rock		

* Based on Refusal in Hole 1B-32

Note: A range in elevation and depths are being provided because of the variance between applicable borings

SOIL PROFILE

Kennedy Interchange
Retaining Wall S9280 (W65-10)
Ramp 2 Stations 41+00 to 41+50
(Based on Hole Nos. 1W-368 and 1W-78)

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description	Parameters	
481.5	0.0			
		Gravel with Sand (Shale Shot Rock) (SC, GP-GM, and GC-GM)	γ_t (lb/ft^3) = 122.0 ϕ (^{o}) = 35.0 k_s (lb/in^3) = 90.0	
				P-Y Curve Reference Number 4
462.5 - 462.5	19.0 - 19.0			
		Lean Clay with Silt and Sand (CL, SM, and SC)	γ_t (lb/ft^3) = 128.0 c_u (tsf) = 0.40 k_s (lb/in^3) = 100.0 E_{50} = 0.007	
				P-Y Curve Reference Number 3
448.5 - 449.5	33.0 - 32.0			
		Sand with Gravel (SP, and SP-SM)	γ_t (lb/ft^3) = 113.0 ϕ (^{o}) = 30.0 k_s (lb/in^3) = 25.0	
				P-Y Curve Reference Number 4
437.5 - 437.0	44.0 - 44.5			
420.0	▽	Sand with Silt and Gravel (SM, SP-SM, SW-SM, and SW)	γ_t (lb/ft^3) = 124.0 γ_e (lb/ft^3)* = 61.6 ϕ (^{o}) = 34.0 k_s (lb/in^3) = 90.0 k_s (lb/in^3) = 60.0	 (above water table) (below water table)
				P-Y Curve Reference Number 4
335.6*	145.9	Top of Rock		

* Based on Refusal in Hole 1B-32

Note: A range in elevation and depths are being provided because of the variance between applicable borings

SOIL PROFILE

Kennedy Interchange
Retaining Wall S9280 (W65-10)
Ramp 2 Stations 41+50 to 42+08
(Based on Hole Nos. 1W-368 and 1W-79)

Approximate Elevation (ft)	Approximate Depth (ft)	STRATA Description	Parameters	
467.5	0.0			
		Gravel with Sand (Shale Shot Rock) (SC, GP-GM, and GC-GM)	γ_t (lb/ft^3) = 122.0 ϕ (°) = 35.0 k_s (lb/in^3) = 90.0	
				P-Y Curve Reference Number 4
462.5 - 462.5	5.0 - 5.0			
		Lean Clay with Silt and Sand (CL, SM, and SC)	γ_t (lb/ft^3) = 128.0 c_u (tsf) = 0.40 k_s (lb/in^3) = 100.0 E_{50} = 0.007	
				P-Y Curve Reference Number 3
449.5 - 449.0	18.0 - 18.5			
		Sand with Gravel (SP, and SP-SM)	γ_t (lb/ft^3) = 113.0 ϕ (°) = 30.0 k_s (lb/in^3) = 25.0	
				P-Y Curve Reference Number 4
437.0 - 435.0	30.5 - 32.5			
			γ_t (lb/ft^3) = 124.0	
420.0	▽	Sand with Silt and Gravel (SM, SP-SM, SW-SM, and SW)	γ_e (lb/ft^3)* = 61.6 ϕ (°) = 34.0 k_s (lb/in^3) = 90.0 k_s (lb/in^3) = 60.0	 (above water table) (below water table)
				P-Y Curve Reference Number 4
335.6*	131.9	Top of Rock		

* Based on Refusal in Hole 1B-32

Note: A range in elevation and depths are being provided because of the variance between applicable borings

P-Y Curve Reference Numbers

1. **Soft Clay with Free Water**. Matlock, H. "Correlations for Design of Laterally Loaded Piles in Soft Clay", *Proceedings*, Offshore Technology Conference, Houston, Texas, 1970, Volume 1, Paper No. 1204, pp. 577-594.

2. **Stiff Clay with Free Water**. Reese, L.C., W.R. Cox, and F.D. Koop, "Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay", *Proceedings*, Offshore Technology Conference, Houston, Texas, Paper No. 2312, 1975, pp. 671-690.

3. **Stiff Clay without Free Water**. Dunnavant, T.W., and M.W. O'Neill, "Performance, Analysis, and Interpretation of a Lateral Load Test of a 72-Inch-Diameter Bored Pile in Over-Consolidated Clay", Department of Civil Engineering, University of Houston-University Park, Houston, Texas, Report No. UHCE 85-4, September, 1985, 57 pages.

4. **Sand Above and Below the Water Table.** Cox, W.R., L.C. Reese, and B.R. Grubbs, "Field Testing of Laterally Loaded Piles in Sand", *Proceedings*, Offshore Technology Conference, Houston, Texas, Volume II, Paper No. 2079, 1974, pp. 459-472.

   American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, API Recommended Practice 2A (RP 2A),* Seventeenth Edition, April 1, 1987.

5. **Soil with Both c and ϕ**. Evans, L.T., and J.M. Duncan, "Simplified Analysis of Laterally Loaded Piles", Report No. UCB/GT/82-04, Geotechnical Engineering, Department of Civil Engineering, University of California, Berkeley, 1982.

6. **Vuggy Limestone (Strong Rock)**. Reese, L.C. and K.J. Nyman, "Field Load Test of Instrumented Drilled Shafts at Islamorada, Florida", a report to Girdler Foundation and Exploration Corporation, Clearwater, Florida, February 28, 1978 (unpublished).

Appendix I

Single H-Pile Capacity
Estimates for Ramp 11
Stations 39+00 to 42+08

Resistance Factors for LRFD*

Driven Piles

Resistance Mechanism	Analysis Methodology	Φ
Axial Capacity		
Skin Friction and End Bearing in Clays	α-Method	0.35
Skin Friction and End Bearing in Sands	Nordlund/Thurman Method	0.45
Uplift Resistance		
Clays	α-Method	0.25
Sands	Nordlund Method	0.35
Axial Capacity - Dynamic Analysis		
Driving Criteria established by dynamic test with signal matching at the beginning of redrive conditions only of at least one production pile per pier, but no less than the number of tests per site provided in Table 10.5.5.2.3-3. Quality control of remaining piles by calibrated wave equation and/or dynamic testing		0.65

Drilled Shafts

Resistance Mechanism	Analysis Methodology	Φ
Axial Capacity		
Side Resistance in Clays	α-Method	0.45
End Bearing in Clays	Total Stress	0.40
Side Resistance in Sands	β-Method	0.55
End Bearing in Sands	SPT Method	0.50
Uplift Resistance		
Clays	α-method	0.35
Sands	β-Method	0.45

* Resistance Factors from AASHTO LRFD Bridge Design Specifications, 4th Edition, Table 10.5.5.2.3-1 for Driven Piles and Table 10.5.5.2.4-1 for Drilled Shafts

Steel H-Pile Capacities
Kennedy Interchanges S9280 (W65-10) Retaining Wall
12x53 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft
Zone contributing to downdrag is based upon a 100 ton pile capacity
Water table at normal pool = 420.0 ft

Soil	Depth Below Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	R_n Total Nominal Geotechnical Axial Resistance (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65) (kips)	(tons)	ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method (kips)	(tons)
Sand	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.1
	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.2
	3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.3
	4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.5
	5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.1	0.6
	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	0.7
Clay	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	0.7
	6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	0.7
	7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	1.0
	8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.8	1.4
	9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.4	1.7
Contributes to Downdrag	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.1	2.1
	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.8	2.4
	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.5	2.7
	13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.1	3.1
	14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.8	3.4
	15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.5	3.7
	16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.2	4.1
	17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.9	4.5
	18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.7	4.8
	19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.4	5.2
	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.5	5.2
Sand	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.5	5.2
	20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.6	5.8
	21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.8	6.4
	21.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	13.0	6.5
	22	2.8	1.4	4.2	2.1	1.9	0.9	2.7	1.4	14.0	7.0
	23	6.3	1.4	7.7	3.8	3.4	1.7	5.0	2.5	15.2	7.6
	24	9.8	1.4	11.2	5.6	5.0	2.5	7.3	3.6	16.5	8.2
	25	13.2	1.4	14.6	7.3	6.6	3.3	9.5	4.8	17.7	8.8
	26	16.7	1.4	18.1	9.1	8.2	4.1	11.8	5.9	18.9	9.5
	27	20.2	1.4	21.6	10.8	9.7	4.9	14.1	7.0	20.1	10.1
	28	23.7	1.4	25.1	12.6	11.3	5.7	16.3	8.2	21.3	10.7
	29	28.0	1.4	29.4	14.7	13.3	6.6	19.1	9.6	22.9	11.4
	30	32.5	1.4	33.9	16.9	15.2	7.6	22.0	11.0	24.4	12.2
	31	36.9	1.4	38.3	19.2	17.2	8.6	24.9	12.5	26.0	13.0
	32	41.4	1.4	42.8	21.4	19.2	9.6	27.8	13.9	27.5	13.8
	33	45.8	1.4	47.2	23.6	21.2	10.6	30.7	15.3	29.1	14.5
	33.1	46.2	1.4	47.6	23.8	21.4	10.7	31.0	15.5	29.2	14.6
	33.1	46.2	7.9	54.1	27.1	24.4	12.2	35.2	17.6	29.2	14.6
	34	53.1	7.9	61.0	30.5	27.5	13.7	39.7	19.8	31.6	15.8
	35	60.8	7.9	68.7	34.3	30.9	15.5	44.6	22.3	34.3	17.2
	36	68.4	7.9	76.3	38.2	34.3	17.2	49.6	24.8	37.0	18.5
	37	76.1	7.9	84.0	42.0	37.8	18.9	54.6	27.3	39.7	19.8
	38	83.7	7.9	91.6	45.8	41.2	20.6	59.6	29.8	42.3	21.2
	39	91.4	7.9	99.3	49.6	44.7	22.3	64.5	32.3	45.0	22.5

Steel H-Pile Capacities
Kennedy Interchanges S9280 (W65-10) Retaining Wall
12x53 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft
Zone contributing to downdrag is based upon a 100 ton pile capacity
Water table at normal pool = 420.0 ft

	Depth Below Pile Cap	Nominal Side Resistance	Nominal End Bearing	R_n Total Nominal Geotechnical Axial Resistance		ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method		ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65)		ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method	
	(ft)	(kips)	(kips)	(kips)	(tons)	(kips)	(tons)	(kips)	(tons)	(kips)	(tons)
Sand	40	99.0	7.9	106.9	53.5	48.1	24.1	69.5	34.8	47.7	23.8
	41	106.7	7.9	114.6	57.3	51.6	25.8	74.5	37.2	50.4	25.2
	42	114.3	7.9	122.2	61.1	55.0	27.5	79.4	39.7	53.1	26.5
	43	123.6	7.9	131.5	65.8	59.2	29.6	85.5	42.8	56.3	28.2
	44	133.2	7.9	141.1	70.5	63.5	31.7	91.7	45.9	59.7	29.8
	45	142.7	7.9	150.6	75.3	67.8	33.9	97.9	48.9	63.0	31.5
	46	152.2	7.9	160.1	80.1	72.1	36.0	104.1	52.0	66.3	33.2
	47	161.8	7.9	169.7	84.8	76.3	38.2	110.3	55.1	69.7	34.8
@48.1 ∇	48	171.3	7.9	179.2	89.6	80.6	40.3	116.5	58.2	73.0	36.5
	49	180.8	7.9	188.7	94.4	84.9	42.5	122.7	61.3	76.3	38.2
	50	190.3	7.9	198.2	99.1	89.2	44.6	128.9	64.4	79.7	39.8
	51	199.9	7.9	207.8	103.9	93.5	46.7	135.1	67.5	83.0	41.5
	52	211.1	7.9	219.0	109.5	98.5	49.3	142.3	71.2	86.9	43.5
	53	222.5	7.9	230.4	115.2	103.7	51.8	149.7	74.9	90.9	45.5
	54	233.9	7.9	241.8	120.9	108.8	54.4	157.2	78.6	94.9	47.4
	55	245.3	7.9	253.2	126.6	113.9	57.0	164.6	82.3	98.9	49.4
	56	256.7	7.9	264.6	132.3	119.1	59.5	172.0	86.0	102.9	51.4
	57	268.1	7.9	276.0	138.0	124.2	62.1	179.4	89.7	106.9	53.4
	58	279.5	7.9	287.4	143.7	129.3	64.7	186.8	93.4	110.9	55.4
	59	290.9	7.9	298.8	149.4	134.5	67.2	194.2	97.1	114.9	57.4
	60	302.3	7.9	310.2	155.1	139.6	69.8	201.6	100.8	118.8	59.4
	61	315.4	7.9	323.3	161.6	145.5	72.7	210.1	105.1	123.4	61.7
	62	328.6	7.9	336.6	168.3	151.5	75.7	218.8	109.4	128.1	64.0
	63	341.9	7.9	349.8	174.9	157.4	78.7	227.4	113.7	132.7	66.4
	64	355.2	7.9	363.1	181.6	163.4	81.7	236.0	118.0	137.4	68.7
	65	368.5	7.9	376.4	188.2	169.4	84.7	244.7	122.3	142.0	71.0
	66	381.8	7.9	389.7	194.8	175.4	87.7	253.3	126.6	146.7	73.3
	67	395.1	7.9	403.0	201.5	181.3	90.7	261.9	131.0	151.3	75.7
	68	408.3	7.9	416.2	208.1	187.3	93.7	270.6	135.3	156.0	78.0
	69	421.6	7.9	429.5	214.8	193.3	96.6	279.2	139.6	160.6	80.3
	70	436.6	7.9	444.5	222.2	200.0	100.0	288.9	144.5	165.8	82.9
	71	451.7	7.9	459.6	229.8	206.8	103.4	298.8	149.4	171.1	85.6
	72	466.9	7.9	474.8	237.4	213.7	106.8	308.6	154.3	176.5	88.2
	73	482.0	7.9	489.9	245.0	220.5	110.2	318.5	159.2	181.8	90.9
	74	497.2	7.9	505.1	252.5	227.3	113.6	328.3	164.2	187.1	93.5
	75	512.3	7.9	520.3	260.1	234.1	117.1	338.2	169.1	192.4	96.2
	76	527.5	7.9	535.4	267.7	240.9	120.5	348.0	174.0	197.7	98.8
	77	542.7	7.9	550.6	275.3	247.8	123.9	357.9	178.9	203.0	101.5
	78	557.8	7.9	565.7	282.9	254.6	127.3	367.7	183.9	208.3	104.1
	79	574.6	7.9	582.5	291.3	262.1	131.1	378.7	189.3	214.2	107.1
	80	591.7	7.9	599.6	299.8	269.8	134.9	389.7	194.9	220.1	110.1

Steel H-Pile Capacities
Kennedy Interchanges S9280 (W65-10) Retaining Wall
14x73 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft
Zone contributing to downdrag is based upon a 140 ton pile capacity
Water table at normal pool = 420.0 ft

	Depth Below Pile Cap	Nominal Side Resistance	Nominal End Bearing	R_n Total Nominal Geotechnical Axial Resistance		**ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method**		**ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65)**		**ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method**	
	(ft)	(kips)	(kips)	(kips)	(tons)	**(kips)**	**(tons)**	**(kips)**	**(tons)**	**(kips)**	**(tons)**
Sand	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.2
	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.3
	3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.5
	4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	0.6
	5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	0.8
	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.9
Clay	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.9
	6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.9	1.0
	7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.7	1.3
	8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.5	1.7
	9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.3	2.1
	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.0	2.5
	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.8	2.9
	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.6	3.3
Contributes to Downdrag	13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.4	3.7
	14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.2	4.1
	15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.0	4.5
	16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.8	4.9
	17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.6	5.3
	18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.5	5.7
	19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.3	6.2
	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.4	6.2
Sand	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.4	6.2
	20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	13.9	6.9
	21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	15.5	7.7
	21.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	15.8	7.9
	22	3.7	2.0	5.7	2.8	2.6	1.3	3.7	1.8	17.1	8.5
	23	8.3	2.0	10.3	5.1	4.6	2.3	6.7	3.3	18.7	9.3
	24	12.9	2.0	14.9	7.4	6.7	3.4	9.7	4.8	20.3	10.2
	25	17.5	2.0	19.5	9.7	8.8	4.4	12.7	6.3	21.9	11.0
	26	22.1	2.0	24.1	12.1	10.8	5.4	15.7	7.8	23.5	11.8
	27	26.7	2.0	28.7	14.4	12.9	6.5	18.7	9.3	25.1	12.6
	28	31.3	2.0	33.3	16.7	15.0	7.5	21.7	10.8	26.8	13.4
	29	37.0	2.0	39.0	19.5	17.6	8.8	25.4	12.7	28.8	14.4
	30	42.9	2.0	44.9	22.4	20.2	10.1	29.2	14.6	30.8	15.4
	31	48.8	2.0	50.8	25.4	22.8	11.4	33.0	16.5	32.9	16.4
	32	54.6	2.0	56.6	28.3	25.5	12.7	36.8	18.4	34.9	17.5
	33	60.5	2.0	62.5	31.2	28.1	14.1	40.6	20.3	37.0	18.5
	33.1	61.1	2.0	63.1	31.5	28.4	14.2	41.0	20.5	37.2	18.6
	33.1	61.0	10.9	72.0	36.0	32.4	16.2	46.8	23.4	37.2	18.6
	34	70.3	10.9	81.3	40.6	36.6	18.3	52.8	26.4	40.4	20.2
	35	80.7	10.9	91.6	45.8	41.2	20.6	59.6	29.8	44.0	22.0
	36	91.0	10.9	102.0	51.0	45.9	22.9	66.3	33.1	47.7	23.8
	37	101.4	10.9	112.3	56.2	50.5	25.3	73.0	36.5	51.3	25.6
	38	111.7	10.9	122.6	61.3	55.2	27.6	79.7	39.9	54.9	27.4
	39	122.1	10.9	133.0	66.5	59.8	29.9	86.4	43.2	58.5	29.3

Steel H-Pile Capacities

Kennedy Interchanges S9280 (W65-10) Retaining Wall

14x73 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft

Zone contributing to downdrag is based upon a 140 ton pile capacity

Water table at normal pool = 420.0 ft

	Depth Below Pile Cap	Nominal Side Resistance	Nominal End Bearing	R_n Total Nominal Geotechnical Axial Resistance		ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method		ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65)		ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method	
	(ft)	(kips)	(kips)	(kips)	(tons)	(kips)	(tons)	(kips)	(tons)	(kips)	(tons)
Sand	40	132.4	10.9	143.3	71.7	64.5	32.3	93.2	46.6	62.1	31.1
	41	142.7	10.9	153.7	76.8	69.2	34.6	99.9	49.9	65.8	32.9
	42	153.1	10.9	164.0	82.0	73.8	36.9	106.6	53.3	69.4	34.7
	43	165.7	10.9	176.6	88.3	79.5	39.7	114.8	57.4	73.8	36.9
	44	178.6	10.9	189.5	94.8	85.3	42.6	123.2	61.6	78.3	39.1
	45	191.5	10.9	202.4	101.2	91.1	45.5	131.5	65.8	82.8	41.4
	46	204.3	10.9	215.3	107.6	96.9	48.4	139.9	70.0	87.3	43.7
	47	217.2	10.9	228.1	114.1	102.7	51.3	148.3	74.1	91.8	45.9
@48.1	48	230.1	10.9	241.0	120.5	108.5	54.2	156.7	78.3	96.3	48.2
	49	243.0	10.9	253.9	127.0	114.3	57.1	165.0	82.5	100.8	50.4
	50	255.9	10.9	266.8	133.4	120.1	60.0	173.4	86.7	105.3	52.7
	51	268.7	10.9	279.7	139.8	125.8	62.9	181.8	90.9	109.8	54.9
	52	283.9	10.9	294.8	147.4	132.7	66.3	191.6	95.8	115.1	57.6
	53	299.3	10.9	310.2	155.1	139.6	69.8	201.6	100.8	120.5	60.3
	54	314.7	10.9	325.6	162.8	146.5	73.3	211.7	105.8	125.9	63.0
	55	330.1	10.9	341.1	170.5	153.5	76.7	221.7	110.8	131.3	65.7
	56	345.5	10.9	356.5	178.2	160.4	80.2	231.7	115.9	136.7	68.4
	57	361.0	10.9	371.9	185.9	167.3	83.7	241.7	120.9	142.1	71.1
	58	376.4	10.9	387.3	193.7	174.3	87.1	251.7	125.9	147.5	73.8
	59	391.8	10.9	402.7	201.4	181.2	90.6	261.8	130.9	152.9	76.5
	60	407.2	10.9	418.1	209.1	188.2	94.1	271.8	135.9	158.3	79.2
	61	424.9	10.9	435.8	217.9	196.1	98.1	283.3	141.6	164.5	82.2
	62	442.8	10.9	453.8	226.9	204.2	102.1	294.9	147.5	170.8	85.4
	63	460.8	10.9	471.7	235.9	212.3	106.1	306.6	153.3	177.1	88.5
	64	478.7	10.9	489.7	244.8	220.4	110.2	318.3	159.1	183.3	91.7
	65	496.7	10.9	507.6	253.8	228.4	114.2	330.0	165.0	189.6	94.8
	66	514.6	10.9	525.6	262.8	236.5	118.3	341.6	170.8	195.9	98.0
	67	532.6	10.9	543.5	271.8	244.6	122.3	353.3	176.6	202.2	101.1
	68	550.6	10.9	561.5	280.7	252.7	126.3	365.0	182.5	208.5	104.2
	69	568.5	10.9	579.4	289.7	260.7	130.4	376.6	188.3	214.8	107.4
	70	588.7	10.9	599.6	299.8	269.8	134.9	389.8	194.9	221.8	110.9
	71	609.2	10.9	620.1	310.1	279.1	139.5	403.1	201.5	229.0	114.5
	72	629.7	10.9	640.6	320.3	288.3	144.1	416.4	208.2	236.2	118.1
	73	650.2	10.9	661.1	330.6	297.5	148.8	429.7	214.9	243.4	121.7
	74	670.7	10.9	681.6	340.8	306.7	153.4	443.0	221.5	250.5	125.3
	75	691.2	10.9	702.1	351.0	315.9	158.0	456.4	228.2	257.7	128.8
	76	711.7	10.9	722.6	361.3	325.2	162.6	469.7	234.8	264.9	132.4
	77	732.1	10.9	743.1	371.5	334.4	167.2	483.0	241.5	272.0	136.0
	78	752.6	10.9	763.6	381.8	343.6	171.8	496.3	248.2	279.2	139.6
	79	775.4	10.9	786.3	393.2	353.8	176.9	511.1	255.6	287.2	143.6
	80	798.4	10.9	809.3	404.7	364.2	182.1	526.1	263.0	295.2	147.6

Steel H-Pile Capacities

Kennedy Interchanges S9280 (W65-10) Retaining Wall
14x89 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft
Zone contributing to downdrag is based upon a 170 ton pile capacity
Water table at normal pool = 420.0 ft

	Depth Below Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	R_n Total Nominal Geotechnical Axial Resistance (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65) (kips)	(tons)	ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method (kips)	(tons)
Sand	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.2
	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.3
	3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0	0.5
	4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	0.7
	5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.7	0.9
	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	1.0
Clay	5.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	1.0
	6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	1.0
	7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.9	1.4
	8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	3.7	1.8
	9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	4.5	2.2
Contributes to Downdrag	10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	5.3	2.6
	11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.1	3.0
	12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.8	3.4
	13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	7.6	3.8
	14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.4	4.2
	15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	9.2	4.6
	16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.1	5.0
	17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	10.9	5.5
	18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	11.8	5.9
	19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.6	6.3
	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.7	6.4
Sand	19.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	12.7	6.4
	20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.3	7.1
	21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	16.0	8.0
	22	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	17.8	8.9
	22.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	18.8	9.4
	23	2.0	2.4	4.4	2.2	2.0	1.0	2.8	1.4	19.5	9.8
	24	7.0	2.4	9.4	4.7	4.2	2.1	6.1	3.0	21.3	10.6
	25	12.0	2.4	14.4	7.2	6.5	3.2	9.3	4.7	23.1	11.5
	26	17.0	2.4	19.4	9.7	8.7	4.4	12.6	6.3	24.8	12.4
	27	22.0	2.4	24.4	12.2	11.0	5.5	15.9	7.9	26.6	13.3
	28	27.0	2.4	29.4	14.7	13.2	6.6	19.1	9.6	28.3	14.2
	29	33.2	2.4	35.6	17.8	16.0	8.0	23.2	11.6	30.5	15.2
	30	39.6	2.4	42.0	21.0	18.9	9.4	27.3	13.6	32.7	16.4
	31	46.0	2.4	48.4	24.2	21.8	10.9	31.4	15.7	34.9	17.5
	32	52.3	2.4	54.7	27.4	24.6	12.3	35.6	17.8	37.2	18.6
	33	58.7	2.4	61.1	30.5	27.5	13.7	39.7	19.9	39.4	19.7
	33.1	59.3	2.4	61.7	30.9	27.8	13.9	40.1	20.1	39.6	19.8
	33.1	59.3	13.3	72.6	36.3	32.7	16.3	47.2	23.6	39.6	19.8
	34	69.6	13.3	82.9	41.4	37.3	18.7	53.9	26.9	43.2	21.6
	35	81.0	13.3	94.3	47.1	42.4	21.2	61.3	30.6	47.2	23.6
	36	92.4	13.3	105.7	52.8	47.6	23.8	68.7	34.4	51.2	25.6
	37	103.8	13.3	117.1	58.5	52.7	26.3	76.1	38.1	55.2	27.6
	38	115.2	13.3	128.5	64.2	57.8	28.9	83.5	41.8	59.2	29.6
	39	126.6	13.3	139.9	69.9	63.0	31.5	90.9	45.5	63.2	31.6

Steel H-Pile Capacities
Kennedy Interchanges S9280 (W65-10) Retaining Wall
14x89 H-Pile (50ksi steel)

Estimated Base of Pile Cap Elevation = 468.1 ft
Zone contributing to downdrag is based upon a 170 ton pile capacity
Water table at normal pool = 420.0 ft

	Depth Below Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	R_n Total Nominal Geotechnical Axial Resistance (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Static Analysis Method (kips)	(tons)	ϕR_n Total Factored Geotechnical Axial Resistance Dynamic Analysis Method (ϕ=0.65) (kips)	(tons)	ϕR_n Total Factored Geotechnical Uplift Resistance Static Analysis Method (kips)	(tons)
Sand	40	138.0	13.3	151.3	75.6	68.1	34.0	98.3	49.2	67.1	33.6
	41	149.4	13.3	162.7	81.3	73.2	36.6	105.8	52.9	71.1	35.6
	42	160.8	13.3	174.1	87.0	78.3	39.2	113.2	56.6	75.1	37.6
	43	174.7	13.3	188.0	94.0	84.6	42.3	122.2	61.1	80.0	40.0
	44	188.8	13.3	202.2	101.1	91.0	45.5	131.4	65.7	85.0	42.5
	45	203.0	13.3	216.4	108.2	97.4	48.7	140.6	70.3	89.9	45.0
	46	217.2	13.3	230.6	115.3	103.8	51.9	149.9	74.9	94.9	47.4
	47	231.4	13.3	244.8	122.4	110.1	55.1	159.1	79.5	99.9	49.9
@48.1	48	245.6	13.3	259.0	129.5	116.5	58.3	168.3	84.2	104.8	52.4
	49	259.8	13.3	273.2	136.6	122.9	61.5	177.6	88.8	109.8	54.9
	50	274.0	13.3	287.4	143.7	129.3	64.7	186.8	93.4	114.8	57.4
	51	288.2	13.3	301.5	150.8	135.7	67.8	196.0	98.0	119.7	59.9
	52	304.9	13.3	318.2	159.1	143.2	71.6	206.9	103.4	125.6	62.8
	53	321.9	13.3	335.2	167.6	150.8	75.4	217.9	108.9	131.5	65.8
	54	338.9	13.3	352.2	176.1	158.5	79.2	228.9	114.5	137.5	68.7
	55	355.9	13.3	369.2	184.6	166.1	83.1	240.0	120.0	143.4	71.7
	56	372.9	13.3	386.2	193.1	173.8	86.9	251.0	125.5	149.4	74.7
	57	389.9	13.3	403.2	201.6	181.4	90.7	262.1	131.0	155.3	77.7
	58	406.8	13.3	420.2	210.1	189.1	94.5	273.1	136.6	161.3	80.6
	59	423.8	13.3	437.2	218.6	196.7	98.4	284.2	142.1	167.2	83.6
	60	440.8	13.3	454.2	227.1	204.4	102.2	295.2	147.6	173.1	86.6
	61	460.3	13.3	473.6	236.8	213.1	106.6	307.9	153.9	180.0	90.0
	62	480.1	13.3	493.4	246.7	222.0	111.0	320.7	160.4	186.9	93.4
	63	499.9	13.3	513.2	256.6	230.9	115.5	333.6	166.8	193.8	96.9
	64	519.7	13.3	533.0	266.5	239.8	119.9	346.4	173.2	200.7	100.4
	65	539.4	13.3	552.8	276.4	248.7	124.4	359.3	179.7	207.7	103.8
	66	559.2	13.3	572.6	286.3	257.7	128.8	372.2	186.1	214.6	107.3
	67	579.0	13.3	592.4	296.2	266.6	133.3	385.0	192.5	221.5	110.8
	68	598.8	13.3	612.1	306.1	275.5	137.7	397.9	198.9	228.4	114.2
	69	618.6	13.3	631.9	316.0	284.4	142.2	410.8	205.4	235.4	117.7
	70	640.9	13.3	654.2	327.1	294.4	147.2	425.2	212.6	243.2	121.6
	71	663.5	13.3	676.8	338.4	304.6	152.3	439.9	220.0	251.1	125.5
	72	686.0	13.3	699.4	349.7	314.7	157.4	454.6	227.3	259.0	129.5
	73	708.6	13.3	721.9	361.0	324.9	162.4	469.3	234.6	266.9	133.4
	74	731.2	13.3	744.5	372.3	335.0	167.5	483.9	242.0	274.8	137.4
	75	753.8	13.3	767.1	383.6	345.2	172.6	498.6	249.3	282.7	141.3
	76	776.4	13.3	789.7	394.8	355.4	177.7	513.3	256.6	290.6	145.3
	77	798.9	13.3	812.3	406.1	365.5	182.8	528.0	264.0	298.5	149.2
	78	821.5	13.3	834.9	417.4	375.7	187.8	542.7	271.3	306.4	153.2
	79	846.6	13.3	859.9	430.0	387.0	193.5	559.0	279.5	315.2	157.6
	80	872.0	13.3	885.3	442.7	398.4	199.2	575.4	287.7	324.0	162.0

Appendix J

H-Pile Driving Resistances

H-Pile Driving Resistance

Kennedy Interchanges S9280 (B65-10) Retaining Wall
Ramp 2 Station 39+00 to 42+08
12x53 H-Pile (50ksi steel)

The driving resistances presented in the table below are based on the following reductions of skin friction during driving:

Clays	-	50%
Sands and Gravels	-	25%

Estimated Base of Pile Cap Elevation = 468.1 ft
Water table at normal pool = 420.0 ft

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	1	0.6	0.6	1.1	0.5
	2	1.3	1.2	2.2	1.1
	3	1.9	1.8	3.3	1.6
	4	2.6	2.4	4.3	2.2
	5	3.3	2.9	5.3	2.7
	5.9	3.9	3.4	6.3	3.1
Clay	5.9	3.8	0.8	3.7	1.8
	6	4.1	0.8	3.8	1.9
	7	6.8	0.8	5.2	2.6
	8	9.5	0.8	6.5	3.3
	9	12.2	0.8	7.9	3.9
Contributes to Downdrag	10	14.9	0.8	9.2	4.6
	11	17.6	0.8	10.5	5.3
	12	20.3	0.8	11.9	5.9
	13	23.0	0.8	13.2	6.6
	14	25.7	0.8	14.6	7.3
	15	28.4	0.8	15.9	8.0
	16	31.3	0.8	17.4	8.7
	17	34.2	0.8	18.9	9.4
	18	37.1	0.8	20.3	10.2
	19	40.1	0.8	21.8	10.9
	19.1	40.4	0.8	21.9	11.0
Sand	19.1	40.4	1.4	22.5	11.3
	20	43.5	1.4	24.9	12.4
	21	47.0	1.4	27.5	13.8
	21.2	47.7	1.4	28.0	14.0
	22	50.5	1.4	30.1	15.1
	23	54.0	1.4	32.7	16.4
	24	57.5	1.4	35.4	17.7
	25	60.9	1.4	38.0	19.0
	26	64.4	1.4	40.6	20.3
	27	67.9	1.4	43.2	21.6
	28	71.4	1.4	45.8	22.9
	29	75.7	1.4	49.1	24.5
	30	80.2	1.4	52.4	26.2
	31	84.6	1.4	55.7	27.9
	32	89.1	1.4	59.1	29.5
	33	93.5	1.4	62.4	31.2
	33.1	93.9	1.4	62.7	31.4
	33.1	93.9	7.9	69.2	34.6
	34	100.8	7.9	74.4	37.2
	35	108.5	7.9	80.1	40.1
	36	116.1	7.9	85.9	42.9
	37	123.8	7.9	91.6	45.8
	38	131.4	7.9	97.3	48.7
	39	139.1	7.9	103.1	51.5

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	40	146.7	7.9	108.8	54.4
	41	154.4	7.9	114.6	57.3
	42	162.0	7.9	120.3	60.1
	43	171.3	7.9	127.3	63.6
	44	180.9	7.9	134.4	67.2
	45	190.4	7.9	141.6	70.8
	46	199.9	7.9	148.7	74.4
	47	209.5	7.9	155.9	77.9
@ 48.1'	48	219.0	7.9	163.0	81.5
	49	228.5	7.9	170.2	85.1
	50	238.0	7.9	177.3	88.7
	51	247.6	7.9	184.5	92.2
	52	258.8	7.9	192.9	96.4
	53	270.2	7.9	201.4	100.7
	54	281.6	7.9	210.0	105.0
	55	293.0	7.9	218.5	109.3
	56	304.4	7.9	227.1	113.5
	57	315.8	7.9	235.6	117.8
	58	327.2	7.9	244.2	122.1
	59	338.6	7.9	252.7	126.4
	60	350.0	7.9	261.3	130.6
	61	363.1	7.9	271.1	135.5
	62	376.3	7.9	281.0	140.5
	63	389.6	7.9	291.0	145.5
	64	402.9	7.9	301.0	150.5
	65	416.2	7.9	310.9	155.5
	66	429.5	7.9	320.9	160.4
	67	442.8	7.9	330.8	165.4
	68	456.0	7.9	340.8	170.4
	69	469.3	7.9	350.8	175.4
	70	484.3	7.9	362.0	181.0
	71	499.4	7.9	373.3	186.7
	72	514.6	7.9	384.7	192.4
	73	529.7	7.9	396.1	198.0
	74	544.9	7.9	407.4	203.7
	75	560.0	7.9	418.8	209.4
	76	575.2	7.9	430.2	215.1
	77	590.4	7.9	441.5	220.8
	78	605.5	7.9	452.9	226.5
	79	622.3	7.9	465.5	232.8
	80	639.4	7.9	478.3	239.2

* Refer to capacity tables and/or construction drawings for required depth/capacities that incorporate other loads that have been accounted for in the design of these piles.

H-Pile Driving Resistance
Kennedy Interchanges S9280 (B65-10) Retaining Wall
Ramp 2 Station 39+00 to 42+08
14x73 H-Pile (50ksi steel)

The driving resistances presented in the table below are based on the following reductions of skin friction during driving:
Clays - 50%
Sands and Gravels - 25%

Estimated Base of Pile Cap Elevation = 468.1 ft
Water table at normal pool = 420.0 ft

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	1	0.9	0.8	1.5	0.7
	2	1.8	1.6	2.9	1.5
	3	2.6	2.4	4.4	2.2
	4	3.5	3.2	5.8	2.9
	5	4.4	4.0	7.3	3.7
	5.9	5.2	4.7	8.6	4.3
Clay	5.9	5.2	1.1	5.0	2.5
	6	5.5	1.1	5.2	2.6
	7	8.7	1.1	6.7	3.4
	8	11.8	1.1	8.3	4.2
	9	14.9	1.1	9.9	4.9
	10	18.1	1.1	11.4	5.7
	11	21.2	1.1	13.0	6.5
	12	24.3	1.1	14.6	7.3
Contributes to Downdrag	13	27.5	1.1	16.1	8.1
	14	30.6	1.1	17.7	8.9
	15	33.7	1.1	19.3	9.6
	16	37.1	1.1	21.0	10.5
	17	40.5	1.1	22.6	11.3
	18	43.8	1.1	24.3	12.2
	19	47.2	1.1	26.0	13.0
	19.1	47.5	1.1	26.2	13.1
Sand	19.1	47.5	2.0	27.1	13.5
	20	51.7	2.0	30.2	15.1
	21	56.3	2.0	33.6	16.8
	21.2	57.2	2.0	34.3	17.2
	22	60.9	2.0	37.1	18.5
	23	65.5	2.0	40.5	20.3
	24	70.1	2.0	44.0	22.0
	25	74.7	2.0	47.4	23.7
	26	79.3	2.0	50.9	25.5
	27	83.9	2.0	54.4	27.2
	28	88.5	2.0	57.8	28.9
	29	94.2	2.0	62.1	31.1
	30	100.1	2.0	66.5	33.2
	31	106.0	2.0	70.9	35.4
	32	111.8	2.0	75.3	37.6
	33	117.7	2.0	79.7	39.8
	33.1	118.3	2.0	80.1	40.1
	33.1	118.2	10.9	89.0	44.5
	34	127.5	10.9	96.0	48.0
	35	137.9	10.9	103.8	51.9
	36	148.2	10.9	111.5	55.8
	37	158.6	10.9	119.3	59.6
	38	168.9	10.9	127.0	63.5
	39	179.3	10.9	134.8	67.4

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	40	189.6	10.9	142.6	71.3
	41	199.9	10.9	150.3	75.2
	42	210.3	10.9	158.1	79.0
	43	222.9	10.9	167.5	83.8
	44	235.8	10.9	177.2	88.6
	45	248.7	10.9	186.8	93.4
	46	261.5	10.9	196.5	98.3
	47	274.4	10.9	206.2	103.1
@ 48.1'	48	287.3	10.9	215.8	107.9
	49	300.2	10.9	225.5	112.7
	50	313.1	10.9	235.1	117.6
	51	325.9	10.9	244.8	122.4
	52	341.1	10.9	256.2	128.1
	53	356.5	10.9	267.7	133.9
	54	371.9	10.9	279.3	139.6
	55	387.3	10.9	290.8	145.4
	56	402.7	10.9	302.4	151.2
	57	418.2	10.9	314.0	157.0
	58	433.6	10.9	325.5	162.8
	59	449.0	10.9	337.1	168.5
	60	464.4	10.9	348.7	174.3
	61	482.1	10.9	361.9	181.0
	62	500.0	10.9	375.4	187.7
	63	518.0	10.9	388.8	194.4
	64	535.9	10.9	402.3	201.2
	65	553.9	10.9	415.8	207.9
	66	571.8	10.9	429.2	214.6
	67	589.8	10.9	442.7	221.4
	68	607.8	10.9	456.2	228.1
	69	625.7	10.9	469.6	234.8
	70	645.9	10.9	484.8	242.4
	71	666.4	10.9	500.2	250.1
	72	686.9	10.9	515.5	257.8
	73	707.4	10.9	530.9	265.4
	74	727.9	10.9	546.3	273.1
	75	748.4	10.9	561.6	280.8
	76	768.9	10.9	577.0	288.5
	77	789.3	10.9	592.4	296.2
	78	809.8	10.9	607.7	303.9
	79	832.6	10.9	624.8	312.4
	80	855.6	10.9	642.1	321.0

* Refer to capacity tables and/or construction drawings for required depth/capacities that incorporate other loads that have been accounted for in the design of these piles.

H-Pile Driving Resistance

Kennedy Interchanges S9280 (B65-10) Retaining Wall
Ramp 2 Station 39+00 to 42+08
14x89 H-Pile (50ksi steel)

The driving resistances presented in the table below are based on the following reductions of skin friction during driving:

Clays	-	50%
Sands and Gravels	-	25%

Estimated Base of Pile Cap Elevation = 468.1 ft
Water table at normal pool = 420.0 ft

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	1	1.0	1.0	1.7	0.9
	2	1.9	1.9	3.4	1.7
	3	2.9	2.9	5.1	2.5
	4	3.9	3.8	6.7	3.4
	5	4.9	4.8	8.5	4.2
	5.9	5.8	5.7	10.0	5.0
Clay	5.9	5.8	1.3	5.6	2.8
	6	6.1	1.3	5.8	2.9
	7	9.3	1.3	7.4	3.7
	8	12.4	1.3	9.0	4.5
	9	15.6	1.3	10.5	5.3
Contributes to Downdrag	10	18.8	1.3	12.1	6.1
	11	21.9	1.3	13.7	6.9
	12	25.1	1.3	15.3	7.6
	13	28.3	1.3	16.9	8.4
	14	31.4	1.3	18.5	9.2
	15	34.6	1.3	20.0	10.0
	16	38.0	1.3	21.7	10.9
	17	41.4	1.3	23.4	11.7
	18	44.8	1.3	25.1	12.6
	19	48.2	1.3	26.8	13.4
	19.1	48.5	1.3	27.0	13.5
Sand	19.1	48.5	2.4	28.1	14.1
	20	53.0	2.4	31.5	15.7
	21	58.0	2.4	35.2	17.6
	22	63.0	2.4	39.0	19.5
	22.6	66.1	2.4	41.3	20.6
	23	68.1	2.4	42.8	21.4
	24	73.1	2.4	46.5	23.3
	25	78.1	2.4	50.3	25.1
	26	83.1	2.4	54.0	27.0
	27	88.1	2.4	57.8	28.9
	28	93.1	2.4	61.5	30.8
	29	99.3	2.4	66.2	33.1
	30	105.7	2.4	71.0	35.5
	31	112.1	2.4	75.8	37.9
	32	118.4	2.4	80.5	40.3
	33	124.8	2.4	85.3	42.7
	33.1	125.4	2.4	85.8	42.9
	33.1	125.4	13.3	96.7	48.3
	34	135.7	13.3	104.4	52.2
	35	147.1	13.3	112.9	56.5
	36	158.5	13.3	121.5	60.7
	37	169.9	13.3	130.0	65.0
	38	181.3	13.3	138.6	69.3
	39	192.7	13.3	147.1	73.6

Soil	Depth Pile Cap (ft)	Nominal Side Resistance (kips)	Nominal End Bearing (kips)	H-Pile Driving Resistance (kips)	H-Pile Driving Resistance (tons)
Sand	40	204.1	13.3	155.7	77.8
	41	215.5	13.3	164.2	82.1
	42	226.9	13.3	172.8	86.4
	43	240.8	13.3	183.2	91.6
	44	254.9	13.3	193.9	96.9
	45	269.1	13.3	204.5	102.2
	46	283.3	13.3	215.1	107.6
	47	297.5	13.3	225.8	112.9
@ 48.1'	48	311.7	13.3	236.4	118.2
	49	325.9	13.3	247.1	123.5
	50	340.1	13.3	257.7	128.9
	51	354.3	13.3	268.4	134.2
	52	371.0	13.3	280.9	140.4
	53	388.0	13.3	293.6	146.8
	54	405.0	13.3	306.4	153.2
	55	422.0	13.3	319.1	159.6
	56	439.0	13.3	331.9	165.9
	57	456.0	13.3	344.6	172.3
	58	472.9	13.3	357.3	178.7
	59	489.9	13.3	370.1	185.0
	60	506.9	13.3	382.8	191.4
	61	526.4	13.3	397.4	198.7
	62	546.2	13.3	412.3	206.1
	63	566.0	13.3	427.1	213.6
	64	585.8	13.3	442.0	221.0
	65	605.5	13.3	456.8	228.4
	66	625.3	13.3	471.6	235.8
	67	645.1	13.3	486.5	243.2
	68	664.9	13.3	501.3	250.7
	69	684.7	13.3	516.2	258.1
	70	707.0	13.3	532.9	266.4
	71	729.6	13.3	549.8	274.9
	72	752.1	13.3	566.7	283.4
	73	774.7	13.3	583.7	291.8
	74	797.3	13.3	600.6	300.3
	75	819.9	13.3	617.5	308.8
	76	842.5	13.3	634.5	317.2
	77	865.0	13.3	651.4	325.7
	78	887.6	13.3	668.4	334.2
	79	912.7	13.3	687.2	343.6
	80	938.1	13.3	706.2	353.1

* Refer to capacity tables and/or construction drawings for required depth/capacities that incorporate other loads that have been accounted for in the design of these piles.